THE UNIVERSITY OF ALBERTA

A STATISTICAL DETERMINATION OF DESIGN FLOOD HYDROGRAPHS

by

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A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

Spring, 1971

UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled

A STATISTICAL DETERMINATION OF

DESIGN FLOOD HYDROGRAPHS

submitted by P.K. Kandasami in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The existing quasideterministic methods of computing the design flood hydrographs involve the drawbacks of unit hydrograph theory. A method was suggested to arrive at the shape of design flood hydrographs by statistical methods. The daily flow records of the North Saskatchewan River at Rocky Mountain House, Edmonton and Prince Albert were used for the analysis.

The various probably rising and recession curves in the form of Q_1, the first day's discharge vs Q_2, the next day's discharge were developed. The median curves obtained were used to study the homogeneity of the population of daily flows, the change in the flood wave with distance travelled downstream and the effect of the record length. The various probably rising curves and the median recession curve together with the flood peak frequency curve were used, to develop the hydrographs of floods of various probabilities. A suggestion was made for the usage of the median recession curve as the characteristic curve of the basin.

ACKNOWLEDGEMENTS

The author wishes to extend his appreciation to Dr. J.P. Verschuren, Professor of Civil Engineering for suggesting this problem and his guidance and constructive criticism throughout this investigation.

Thanks are also due to the National Research Council of Canada for the financial assistance provided. The author is thankful to his friends who helped in preparing this thesis.

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GLOSSARY OF SYMBOLS

D - Discharge level in CFS

C - A constant

K_r - Recession coefficient

M - The total of years in the series from which the samples are taken

N - The number of years in which the events are considered for sampling

P - Probability

Q_1 - Mean discharge on the first day of the consecutive two days under consideration

Q_2 - Mean discharge on the second day of the consecutive two days under consideration

Q_a - Annual discharge

Q_{am} - Annual mean discharge

Q_{mean} - The average of Q_1 and Q_2

Q_{ratio} - The ratio of Q_1 to Q_2

S - Sample standard deviation

Y - The year of occurrence of Q_a

n - The number of samples of N years of record derived from a series of M years record

q_o - Discharge at any time instant

q_t - Discharge t time units after occurrence of q_o

x - The magnitude of half the difference between Q_1 and Q_2

y - The percent deviation of Q_{ratio} - median

CHAPTER I

INTRODUCTION

1.1 General

The science of hydrology today includes a very wide range of topics, offering many diverse fields for investigation. One of the first of these to be studied still perhaps constitutes the principal general problem, namely the need for a reliable means of predicting the response of a catchment to an input of rainfall or snowmelt. A solution would permit reliable flood forecasts to be made, accurate estimates of catchment yield to be established and efficient design figures to be used in engineering construction. Since the problem involves the analysis of complex natural phenomena the perfect solution will always appear elusive; but constant improvement of the techniques will lead not only to better estimates of the values sought, but also to an improved understanding of the problem itself.

1.2 Runoff of a Catchment

The outflow from a catchment is usually represented as a runoff hydrograph combined with the hyetograph. The important characteristics of this representation are the time interval between rainfall and runoff, the volume of runoff produced for given amount of rainfall and the time distribution of this volume. These three factors are all inter-related in a manner which is not completely understood as yet, but in an attempt to analyse the

hydrograph they are often investigated in a comparative isolation.

The work described in this thesis is primarily concerned with the second and third factors; the volume of runoff produced and the time distribution of this volume.

1.3 Outline of the Investigation

The investigation carried out for the project has been presented in the following four chapters.

Chapter II gives a brief description of the runoff hydrograph concerning its shape, components and the factors affecting them. This provides a variety of definitions of the terms involved.

A survey of the literature relevant to flood hydrology is given in Chapter III. This chapter gives the objectives of the present study.

Chapter IV gives a brief description of the statistical methods applied in predicting the design flood hydrograph. In this chapter the present method developed for the computation of design flood hydrographs has been explained. It gives the description of the basin under study.

Chapter V presents the results and discussion on the present study.

Chapter VI gives the conclusion drawn based on the present study.

1.4 Notations

Symbols used in this thesis are defined where they first appear and are collected for convenient reference in 'The Glossary of Symbols'.

CHAPTER II

GENERAL DESCRIPTION OF THE HYDROGRAPH

2.1 Definition of Hydrograph

A hydrograph can be regarded as an integral expression of the physiographic and climatic characteristics that govern the relations between rainfall and runoff of a particular drainage basin. It shows the time distribution of runoff at the point of measurement defining the complexities of the basin characteristics by a single empirical curve.

2.2 Description of Hydrograph

A typical hydrograph produced by a concentrated storm rainfall is a single-peaked skewed curve; multiple peaks may appear on a hydrograph indicating abrupt variation in rainfall intensity, a succession of storm rainfalls, or other causes. A typical single peaked hydrograph (FIG. 2.1) consists of three parts: the approach segment (limb or curve) AB, the rising (or concentration) segment (limb or curve) BD, and the recession (falling) segment (limb or curve) DH. The lower portion of the recession segment is a ground water recession (or depletion) curve which shows the decreasing rate of ground water inflow. On these segments are shown the point of rise B, two points of inflection C and E, the peak point D and two other characteristic points F and G denoting the changes in slope of log (discharge) plotted against time. The

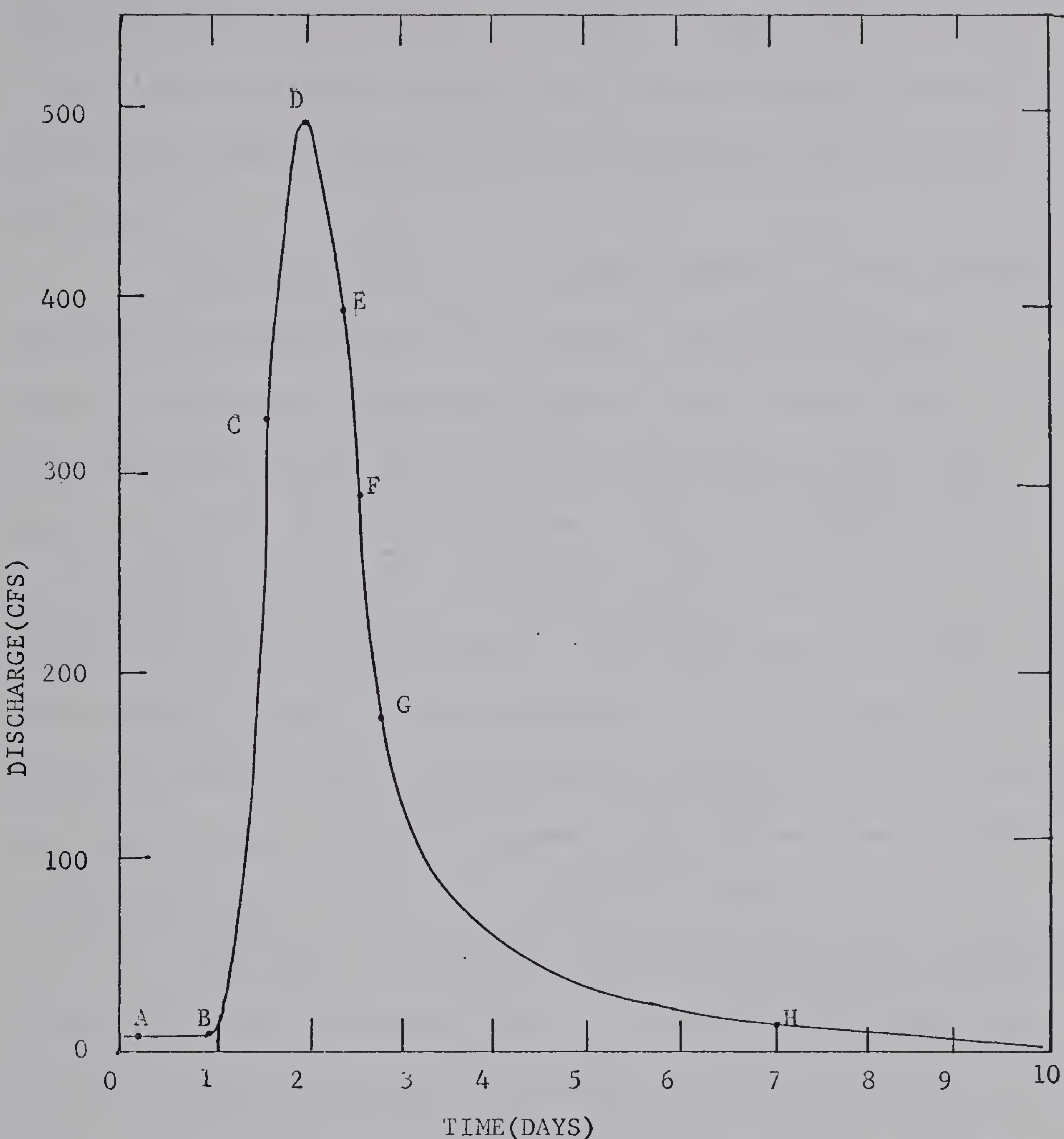


FIG. 2.1 TYPICAL SINGLE PEAKED HYDROGRAPH.

segment CE is the crest segment. The time at point B is the time of rise, at D is the time of peak flow, and from centre of mass of rainfall to centre of mass of runoff is variously defined as the lag time.

The shape of the rising segment depends on the duration and intensity distribution of the rainfall, and the antecedent moisture condition and time-area diagram of the drainage basin. For a dry antecedent ground condition, a uniform rainfall will loose most of its water through abstractions. Thus, the effective rainfall will increase steadily, causing a steadily increasing rate of runoff or a steadily rising segment. For most basins, the time-area diagram is close to a pear shape with the area between isochrones being largest in the middle or upstream part of the basin. This tends to cause the rising segment to be concave upward, rising gradually and then, rapidly toward the end of rise.

The peak of a hydrograph represents the highest concentration of the runoff from a drainage basin. It occurrs usually after the rain has ended and the time of occurrence depends on the areal distribution of the rainfall and other factors. However, if the storm pattern is of an advanced type with a diminishing rainfall intensity, the peak may occur before the end of the rainfall. Hydrographs with multiple peaks may occur in any basin as the result of multiple storms being developed close to each other. If a hydrograph shows double or triple peaks, fairly regularly, the reason may be due to either non-synchronization of the runoff contributions

from several tributaries to the main stream, or a kidney-like shape of the drainage basin.

The recession segment represents the withdrawal of water from storage after all inflow to the channel has ceased. Therefore, it is dependent to a lesser degree, on areal rainfall distribution and to a greater degree on soil moisture conditions and physiographic characteristics of drainage basin.

2.3 Factors Affecting the Hydrograph

Of the two major groups of factors that influence the runoff from a drainage basin the climatic factors include mainly the effects of various forms and types of precipitation, in accordance with the climatic environment. Physiographic factors may be further classified into two kinds: basin characteristics and channel characteristics. Basin characteristics include such factors as size, shape and slope of drainage area, permeability and capacity of groundwater formations, presence of lakes and swamps, and land use. Channel characteristics are related mostly to hydraulic properties of the channel which govern the movement of stream flows and determine channel-storage capacity. It should be noted, however, that the above classification of factors is by no means exact because many factors are interdependent to a certain extent.

The factors affecting runoff generally tend to cause most large drainage basins to behave differently from most small drainage areas on the basis of hydraulic behaviour. Consequently, drainage basins may be classified as large and small, not on the

basis of size alone, but on the effects of certain dominating factors. Frequently, two basins of nearly the same size may behave entirely differently in runoff phenomena. One drainage basin may show prominent channel storage effects like most large basins, while the other may manifest strong influence of land use, like most small basins. A small drainage basin may be defined as one that is so small that its sensitivity to high-intensity rainfalls of short durations and to land use is not suppressed by the channel characteristics. According to this definition, the size of small basins may be from a few acres to 100 acres, or even up to 100 square miles.

2.4 Components of the Hydrograph

Logical and qualitative analysis of observed data indicates the existence of four hydrograph components: surface runoff, subsurface runoff, ground-water runoff, and runoff from precipitation directly on the water bodies -- each of which reaches the stream by a different path. It is practically impossible to measure the volumes of water following each path or to identify the water once it reaches the channel.

That water which reaches the channel by travelling over the soil surface is called surface runoff. This component of runoff is the residual after infiltration, interception, and surface storage have been extracted from precipitation. This runoff is generally considered to be the most important component of flow in major floods.

The bulk of the surface runoff is discharged into the streams during the period of rain, while other components of flow reach the streams much later and are spread over a longer period of time. The portion of water that infiltrates the soil surface and moves laterally through the upper soil horizons until its course is intercepted by a stream channel or until it returns to the surface at some point downslope from its point of infiltration, is known as subsurface flow or interflow. Whenever the soil in the zone of aeration contains sufficient moisture to permit the passage of gravity water downward, a portion of the rainfall reaches the groundwater table. The amount of water which takes this path is limited to that which can percolate through the least permeable soil horizon during the period that free water is available in the soil. Since groundwater flow is distributed over a long period of time, its changes are gradual and groundwater does not represent an important contribution to flood peaks. The fourth source of stream is that precipitation which falls directly on the water surfaces of lakes and streams and this component is known as channel precipitation. Stream flow from this source may be computed by multiplying the average rainfall by the percentage of the basin area covered by water surfaces connected with stream system, but since it is a relatively small amount, it is usually included with surface runoff.

CHAPTER III

LITERATURE REVIEW AND THE PROBLEM

3.1 Historical Resumé of Studies on Flood Hydrology

As far as is known the first substantial investigation in the field of flood hydrology was performed in 1599 by the Italian architect Fontana da Meli who presented to Pope Clement VIII a comprehensive report on a major flood that occurred in the river Tiber in 1598 (Biswas 1967). Little advance seems to have been made in the art of flood hydrology for two-and-a-half centuries following da Meli's historic report.

The enunciation, in the middle of the nineteenth century, of the Rational Method for estimating peak runoff marked the beginning of modern flood hydrology. According to Dooge (1967) it was Mulvaney who published the first report on this technique (in the Transactions of the Institute of Civil Engineers of Ireland, 1851). In estimating a design flood, the need to specify frequency of occurrence gained wide recognition during the first quarter of the current century and since 1914 there have been many developments in the domain of flood frequency analysis.

In a report published in 1930 the Committee on Floods of Boston Society of Civil Engineers pointed to the importance not only of the peak value but also of the time distribution of flood discharge. L.K. Sherman, himself a member of the original Committee on Floods, developed these ideas and, in 1932, enunciated his classical

unit hydrograph theory.

There appears to have been no startling advances in flood hydrology in the period from the early 1930's to the late 1950's, with accumulation of practical experience during these years of consolidation, established methods were improved, theories were refined and empirical factors evaluated. Probably the most important contributions during this period was Bernard's (1935) distribution graph and the S-hydrograph, introduced by Morgan and Hullinghorst (1939).

It was not until the late 1950's that the output from some early theoretical conceptual models of drainage systems, based on a time-area-concentration diagram proposed many years earlier by Ross (1921), was recognized as having the form of an instantaneous unit hydrograph. Full descriptions by O'Kelly (1955) and Nash (1957) of the instantaneous unit hydrograph concept paved the way for a variety of mathematical (as opposed to intuitive and empirical) analyses of flood response. In most mathematical analyses the instantaneous unitgraph is represented by the kernel function of a convolution integral - also known as the Duhamel integral (Dickenson et al 1967.)

Whereas since 1940 there have appeared many linear storage models of natural drainage systems, all based on some form of the time-area diagram, Laurenson (1962) produced one of the earliest catchment models embodying non-linear storage components. The work of Amorocho (1963) on functional series represents a wide

front of research effort currently directed towards development of analytical methods employing sophisticated non-linear mathematical models of stream flow response. The dearth of high quality hydrological data is probably the greatest obstacle to the general application of such methods in practical design flood determination.

During the 1960's a great deal of effort has been directed towards development of catchment response models capable of simulating, with some degree of success, the moisture budget in the runoff cycle. The Tennessee Valley Authority (1965) developed the so-called dynamic watershed model and more recently Midgley and Schultz (1965), Crawford and Linsley (1966) and Schultz (1968) succeeded in synthesizing runoff from rainfall by lag-and-route storage models. Refinement of synthesis models is still hampered by lack of high quality hydro-meteorologic data as well as by incomplete understanding of the physical process in the runoff cycles; as soon as these technical problems can be surmounted, synthesis models will without doubt play an important role in flood hydrograph determination. The Rational Method is perhaps the most popular method of design flood determination. If operated with satisfactory rainfall depth/duration/frequency data and carefully chosen time parameters and catchment coefficients, the method can be highly acceptable. Considerable attention has been paid to the frequency of flood peak discharges. Increased use is being made of experience envelopes as a basis for determination of design floods. Many of the methods used by engineers however

provide an estimate of the peak discharge only whereas for many design purposes the full hydrograph is needed. Pullen (1966) adopted a method to develop unit graphs for an ungauged point on a river whose basin has no recording rain gauges.

The branches of mathematics - probability theory, mathematical statistics, stochastic process, systems analysis - have made substantial advances during the past two decades. If hydrology is to progress it should take the advantage of adopting these techniques. Considerable amount of effort has been put in applying these techniques in forecasting the design flood from a drainage basin. A quasideterministic method, developed using unit graph theory, by Pullen (1969) and Midgley et al (1969) could be used to predict the runoff. The unit hydrograph theory enunciated by L.K. Sherman was stated by himself as "The unit hydrograph is hydrograph of runoff from a given drainage basin, due solely to the volume of net rainfall (precipitation excess) occurring in a specified unit of time." The following are the main assumptions of unit graph theory:

a) Causative rainfall is uniformly distributed over the catchment and occurs at a constant uniform rate.

b) Catchment response to storm input is linear and independent of antecedent conditions - i.e. the principle of superposition is applicable.

c) Corresponding ordinates of storm runoff hydrographs produced by excess rainfalls of equal duration are proportional to volume of

excess rainfall.

Criticisms and alternative forms of these assumptions can be found in the excellent bibliography compiled by Dickinson, Holland and Smith (1967). Most text books on hydrology offer a chapter on unit graph technics; Johnstone and Cross (1959) in particular give a lucid description of the underlying principles and discuss many of the derivatives of unit graph theory.

All established methods of unit graph derivation, except the trial and error method proposed by Barnes (1959), rely on concurrent recordings of both rainfall and runoff. Coulter (1961) showed that procedural differences in deriving unit graphs have little effect on resulting estimates of flood discharge.

3.2 The Present Study

The present analysis is an attempt to develop hydrographs for floods of required frequency using statistical techniques. The daily discharge records of the North Saskatchewan River at Rocky Mountain House, Edmonton, and Prince Albert, were used for the present investigation. The computations were carried out at the University of Alberta IBM 360/67 Computer, and most of the plotting was done by the use of the software routines with the model 770/663 calcomp plotter.

The objectives of the present investigations are outlined in the following paragraphs.

1. To develop, using statistical methods, the median rising and recession curves of the hydrograph, and to represent these

curves as graphs of the discharge Q_1 on a day versus the discharge Q_2 on the following day.

2. To study the variation of the aforementioned median curves estimated from shorter records from those obtained from longer records considered for the present study, and to determine the minimum record length required to yield results within certain reasonable deviations.

3. To consider an overlapping period of required minimum record length for some stations (preferably some upstream and some downstream of the station under consideration) and to obtain the median curves for all these stations and to study whether the relative differences in the curves obtained statistically bear any hydraulic and hydrologic reasonings.

4. To split the flow record of a gauging station, obtain the median curves for different periods of record, and test for the population homogeneity. Also, to test the randomness of annual flow by studying its trend.

5. To develop the rising and recession curves of various probabilities, and use these curves to obtain statistically the hydrographs of required probable floods, and to compare the expected hydrographs of floods of given recurrence intervals with the hydrographs of historical floods of the corresponding recurrence intervals.

CHAPTER IV

ANALYSIS OF HYDROGRAPH

4.1 Statistical Methods in Flood Hydrology

It is important to appreciate that a design flood of given recurrence interval may result from an infinite number of combinations of storm precipitation and catchment state, e.g. at one extreme a severe storm much rarer than the design flood and occurring when the catchment has been saturated by prolonged relatively light rains, or, at the other extreme, a storm of moderate intensity occurring as an isolated event when the catchment is in a relatively dry state. Since both input and catchment state are stochastic in character and are reasonably uncorrelated, the probability of occurrence of the resulting output would be approximately the product of the probabilities of occurrence of the particular input and catchment state at the time of the event. In the face of this infinite range of joint probabilities it is conventional and convenient to base estimation of design flood discharge of a particular event having the recurrence interval as that of the design flood and to seek values for the catchment parameters that have maximum likelihood of occurring under design conditions. When endeavouring to estimate probable maximum flood discharge (PMF), however, it follows that probable maximum precipitation (PMP) must be coupled with extreme values of the catchment

state parameters, i.e. probable minimum loss rates.

Despite the fact that relatively well defined probability distributions of storm rainfall can be derived from recorded data, combination of these with rather poorly defined probability distributions of the catchment state parameters that affect loss rate must inevitably impair the confidence with which the resulting flood discharge can be estimated. Nevertheless, it is contended that this quasi-deterministic approach to design estimation is superior to one based purely on frequency analyses of recorded flood peaks.

But the assumptions involved in the unit graph theory, used in the quasi-deterministic approach, are seldom met in practice. For larger basin area covering varying climatic and physiographic characteristics the assumption of uniformity with respect to both area and time is doubtful. The catchment response to storm input has been found to be non-linear in nature and hence the principle of superposition is not exact. Consequently, the unit graph technique is applicable with reasonable justification only when the precipitation excess is close to one inch depth.

For these reasons it is attempted (in this thesis) to determine expected hydrographs of floods of a desired return period, with the aid of daily discharge records of the river. The treatment is purely statistical in nature, and the following assumptions are involved.

1) Hydrographs of high peaked floods are associated with higher rate of increase of discharge for the rising period.

2) The recession curve is a characteristic curve of the given basin.

3) The events considered as samples are independent and the population is homogeneous.

With the aforementioned assumptions the analysis was carried out as described in the following section.

4.2 Analysis of Daily Discharges

The daily discharge records of the North Saskatchewan River for the three stations under consideration stretch over a period of 48 years (1918 through 1965) for Edmonton, and 13 years each (1918 through 1930) for Rocky Mountain House and Prince Albert. The reason to use the record starting since the year 1918 is that the records obtained for the earlier periods were not clear and continuous. The details of the gauging stations on the river are presented in Table IV.1. Fig. 4.1 shows the catchment area, upstream of Edmonton, with the main river and its tributaries. For the period of record considered for Edmonton, the average daily discharge and its standard deviation were computed to be 7524 cfs and 8552 cfs respectively. The instantaneous maximum flow for the period is 132000 cfs that occurred on the 25th June, 1952. A still higher instantaneous peak of 204500 cfs has occurred on the 29th June, 1915, but this lies outside the period of record under consideration. The discharge at Edmonton is affected by regulations at the Brazeau River Power Development; but in the present analysis the portion of the record used after the construction of the plant is insignificant.

TABLE IV.1

GAUGING STATIONS ON THE NORTH SASKATCHEWAN RIVER.

PLACE AND CODE OF STATION	LOCATION	DRAINAGE AREA (sq.mls)	GAUGE TYPE	REMARKS
Saskatchewan-Crossing 5DA-G	Lat 51° 58' 00" Long 116° 43' 30"	492	Recording	Records fair.
Tershishner Creek 5DC-7	Lat 52° 18'00" Long 116° 19' 00"	1430	Recording	Data supplied by Calgary Power Ltd.
Saunders 5DC-2	Lat 52° 27' 10" Long 115° 45' 20"	1980	Recording	Records good except for periods of estimated flows which are fair.
Rocky Mountain-House 5DC-1	Lat 58° 22' 05" Long 114° 45' 20"	4220	Recording	Records good during open water period & fair during ice period.
Edmonton 5DF-1	Lat 53° 32' 20" Long 113° 29' 10"	10500	Recording	Records fair.
Lea Park	Lat 53° 39' 30" Long 110° 20' 20"	21300	Recording	Records fair.
Prince Albert	Lat 53° 12' 10" Long 110° 20' 20"	46100	Manual	Records good during open water period & fair during ice period.

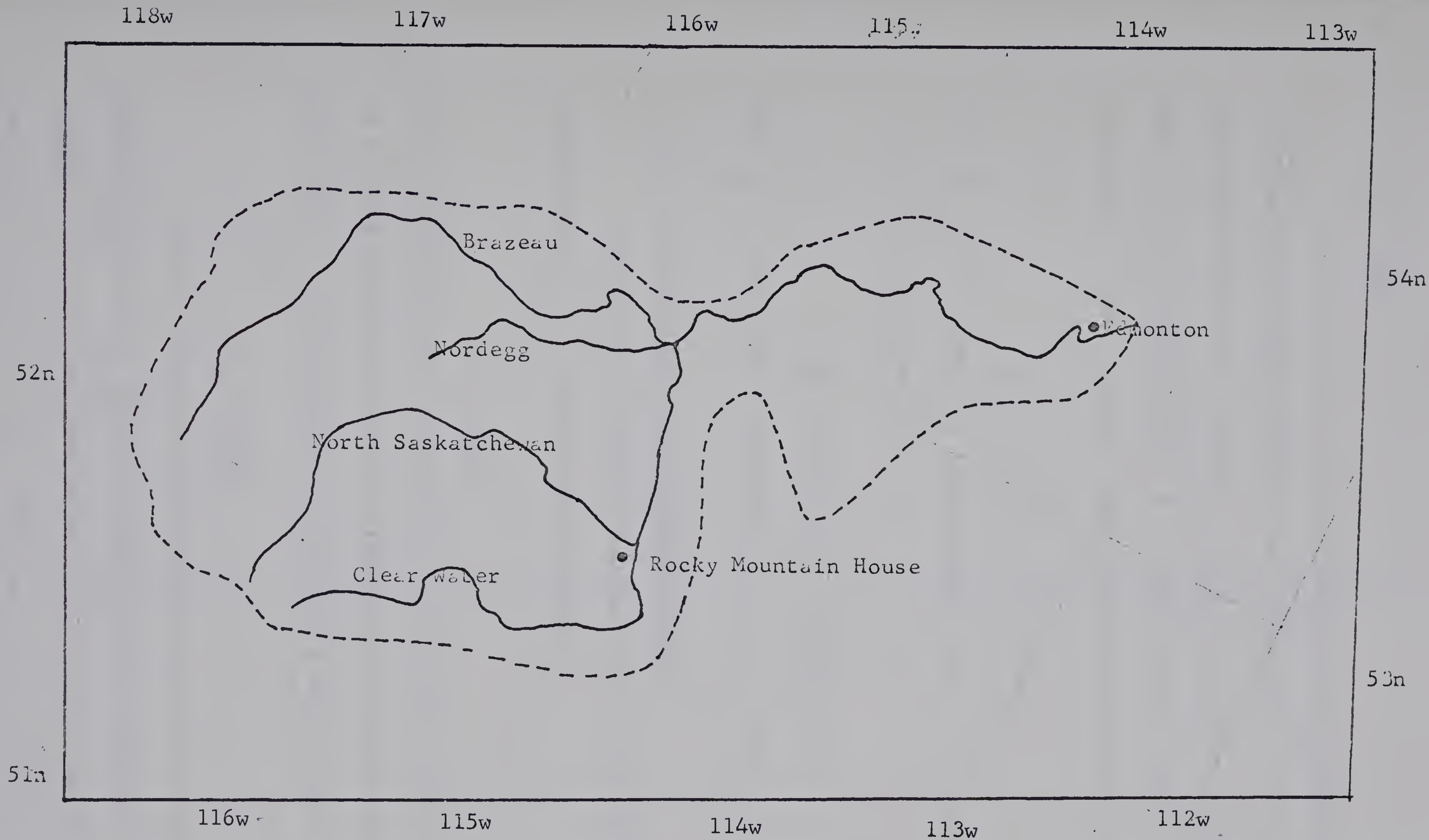


FIG. 4.1 MAP OF THE NORTH SASKATCHEWAN RIVER BASIN UPSTREAM OF EDMONTON.

4.3 Trend Study

Since the flow records considered for Edmonton span over a comparatively longer period it was subjected to trend study. The annual discharges were computed and plotted against the corresponding years. This is shown in Fig. 4.2. The best fit line was drawn using the method of least squares. The mean line represented by

$$Q_{a.m} = 27.55 \times 10^5 \text{ CFS.day} \qquad 4.1$$

where $Q_{a.m}$ is the annual mean discharge, seems to be approximate to the best fit line given by

$$Q_a = 26.25 \times 10^5 + 5304(Y-1917) \qquad 4.2$$

where Q_a is the annual discharge and Y is the year of occurrence.

4.4 An[illegible]sis of Rising and Recession Curves

Linsley et al (1958) described the representation of the recession curve by a characteristic equation

$$q_t = q_o K_r^{\,t} \qquad 4.3$$

where q_o is the flow at any time, q_t is the flow t time units later, and K_r is a constant called the recession coefficient which is less than unity. If the time unit is taken as 1 day, equation 4.3 becomes

$$q_1 = q_o K_r \qquad 4.4$$

On small basins a shorter time unit may be necessary. The numerical value of K_r depends on the time unit selected. Since in the analysis only daily mean discharges are available the time unit can be chosen only in terms of days. The time unit selected was one day.

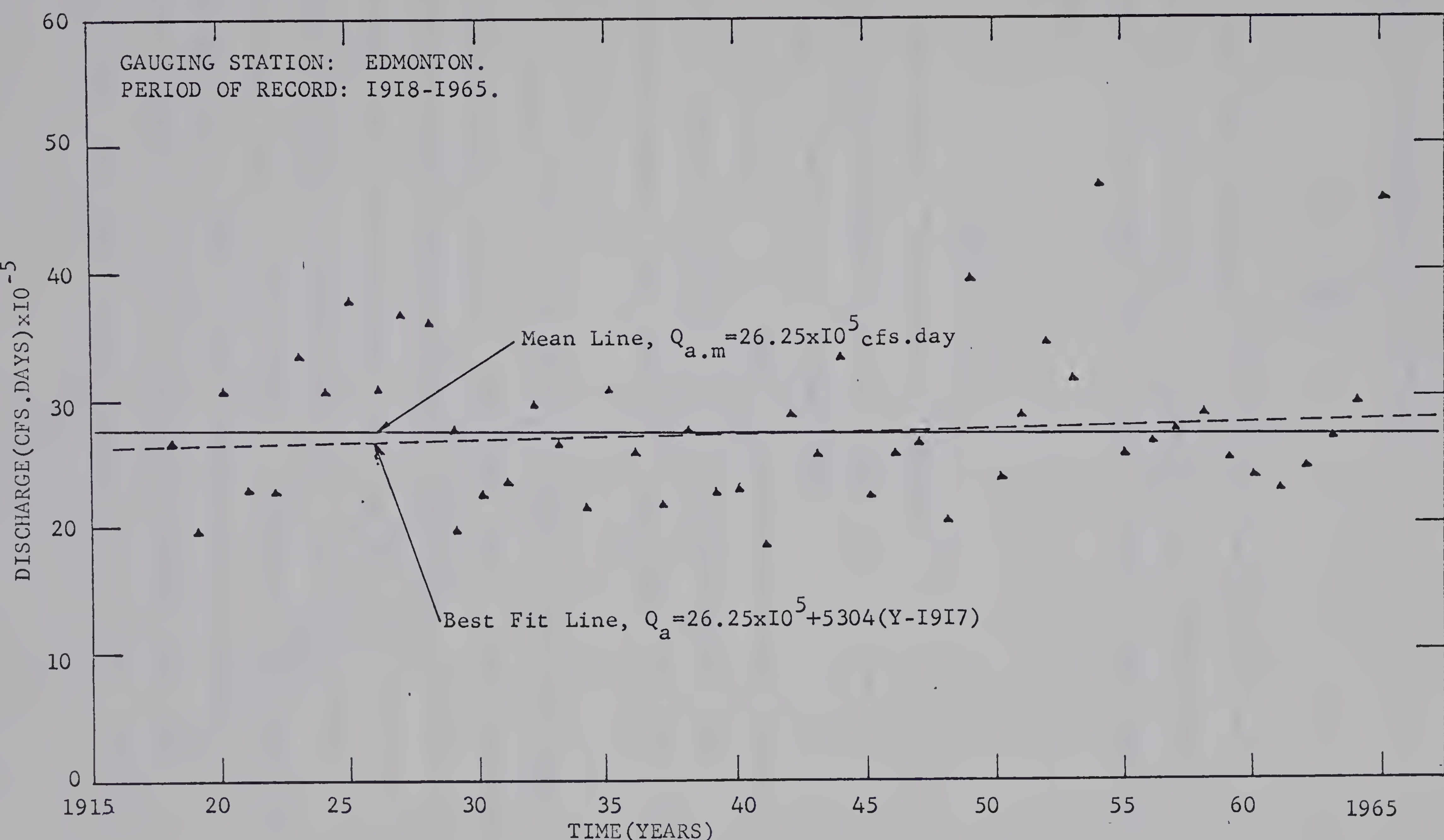


FIG. 4.2 ANNUAL FLOWS OF THE NORTH SASKATCHEWAN RIVER AT EDMONTON vs YEAR.

The equation 4.4 can be written in the form,

$$\frac{1}{K_r} = \frac{Q_1}{Q_2} \qquad 4.5$$

where Q_1 is the mean discharge of the first day and Q_2 is the mean discharge of the following day. In this study $1/K_r$ is denoted by Q_{ratio} which is always greater than or equal to unity. Thus, equation 4.5 becomes,

$$Q_{ratio} = Q_1/Q_2 \qquad 4.6$$

This type of representation - the plot of Q_1 vs Q_2 - for the recession curve is shown in Fig 4.4. This curve is given for the flow data, at Edmonton, for the year 1918.

It was proposed to attempt to describe the rising curve by equation 4.6. Here the value of Q_{ratio} is less than unity. The rising curve developed in this form for the flow data of the year 1918 is shown in Fig 4.3.

To analyse the rising events, a value for daily discharge was considered and was designated as discharge level D. All the events where any pair of two consecutive days had their discharge levels such that the first day's discharge Q_1 was less than the level D and the second day's discharge Q_2 was above the level, were considered as samples of such events in the population. Samples were thus obtained for discharge levels of 1000 cfs and 5000 cfs through 60000 cfs with increments of 5000 cfs. Since the sample size becomes very small for levels higher than 60000 cfs the sampling was abandoned. These

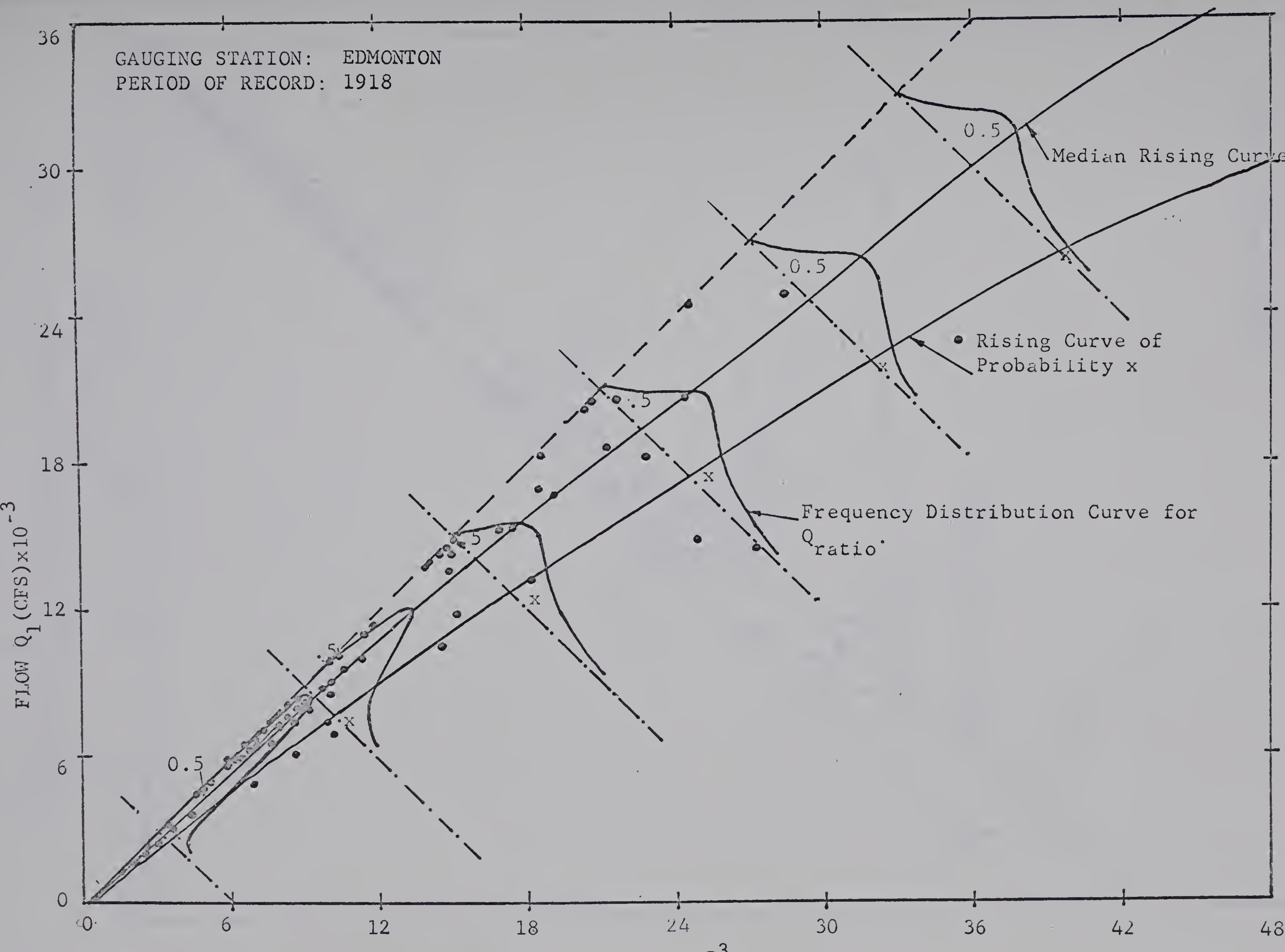


FIG. 4.3 RISING CURVE IN THE FORM OF Q_1 vs Q_2.

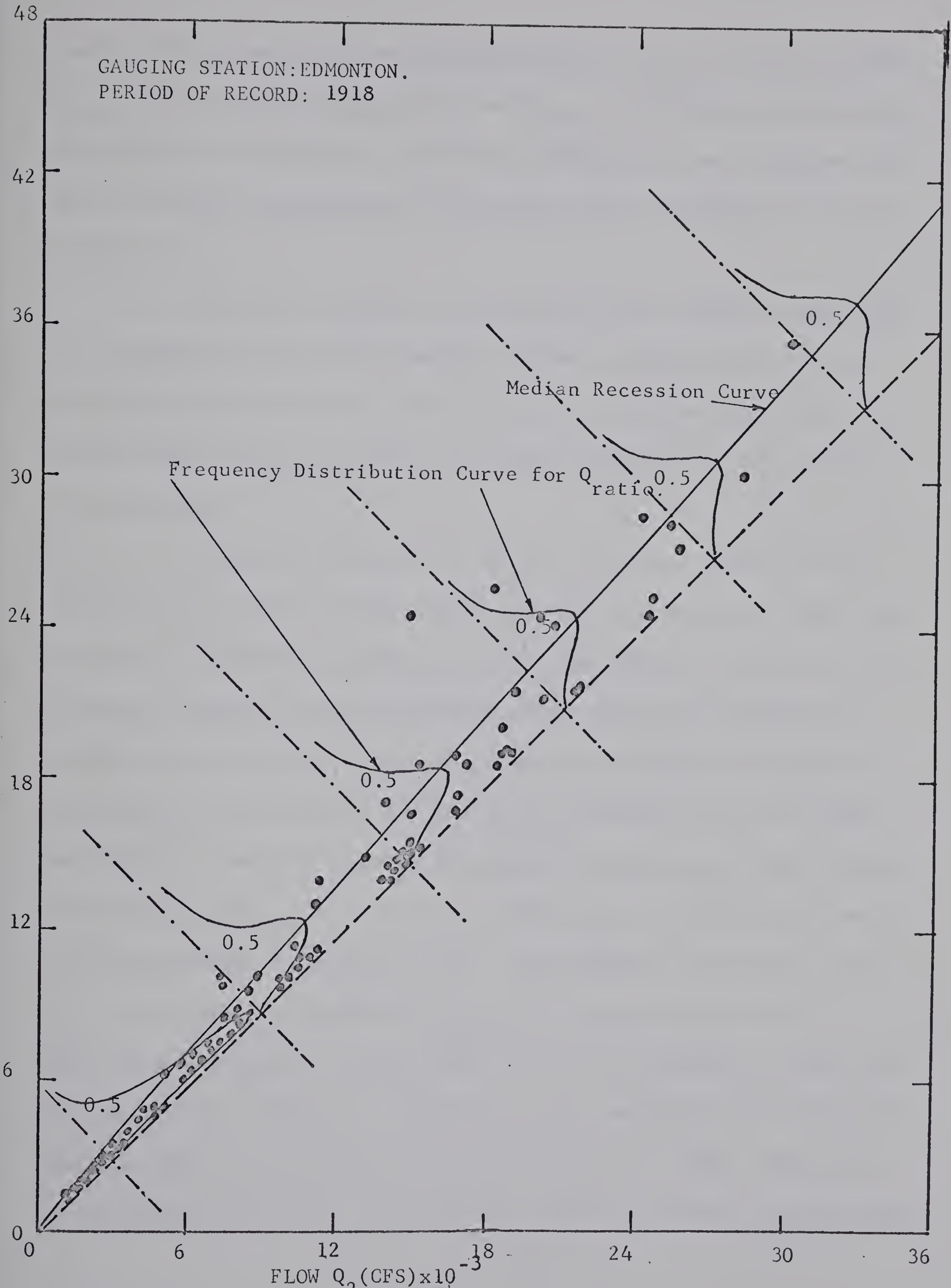


FIG. 4.4 RECESSION CURVE IN THE FORM OF Q_1 vs Q_2.

samples were treated for two variables namely, Q_{ratio} and Q_{mean} where Q_{mean} is the average of Q_1 and Q_2 and Q_{ratio} has already been defined. A correlation analysis was carried out between the two variables for all the levels considered and its results will be discussed later in chapter 5.

The above analysis was carried out in a similar manner for the recession cases except that the samples came from those events where the discharge on the first day of the two consecutive days was greater than the given level D, and that on the second day was less than the level.

Frequency histograms of Q_{ratio} for the discharge levels of 1000 cfs and 10000 through 60000 cfs with increments of 10000 cfs are given in Appendix A, Figs A1 to A7. The similar Figures for the discharge levels of 5000 cfs through 55000 cfs with increments of 10000 cfs have not been presented because the change in frequency histogram for an increment of 5000 cfs in discharge level was found negligible. Similar frequency histograms for the Q_{mean}, after being normalized as $(Q_{mean}-D)/S$ where S is the sample standard deviation are given in Appendix B, Figs B1 to B7. The cumulative frequency curves i.e. the probability curves of Q_{ratio} for the discharge levels of 1000 cfs and 5000 cfs through 60000 cfs with an increment of 5000 cfs are presented in Appendix C, Figs C1 to C13. Here the curves for all the discharges considered are presented because of their importance in developing the rising and recession curves of various probabilities.

4.5 Development of Rising and Recession Curves.

Figs. 4.3 and 4.4 give a picture of how rising and recession events represented in the form of Q_1 and Q_2 appear. The figures show larger scattering of points about the median curve in the rising cases than in the recession cases. An observation on similar plots obtained for all the 48 years showed the same characteristics. All the median curves drawn for the flows of each year showed more consistency in the recession characteristics than in the rising characteristics of the hydrograph. From these facts it is clear that the recession curve depends only on the basin characteristics and not on the precipitation characteristics.

In the following lines, the method to develop the rising and recession curves of different probabilities will be explained. From the probability curves (Appendix C) for Q_{ratio} of rising cases, the different probable Q_{ratio}-values were obtained and presented in Table D.1A (Appendix D). Assuming that Q_1 and Q_2 are by an equal amount x respectively below and above D, equation 4.6 can be rewritten as

$$\frac{D - x}{D + x} = Q_{ratio} \qquad 4.7$$

The above equation can be solved for the values of x and hence for the values of Q_1 and Q_2. The values of Q_1 and Q_2 obtained for considered values of discharge levels and the required probable Q_{ratio} values are given in Table D.2A (Appendix D).

The same procedure was repeated for the recession cases

except that equation 4.7 becomes

$$\frac{D + x}{D - x} = Q_{ratio}$$

Although, only the median recession curve is needed for the present investigation other probable curves as in the rising cases were also developed. The necessary probable Q_{ratio} values are obtained from the figures in Appendix C and the obtained values of Q_{ratio} and the computed values of Q_1 and Q_2 are presented respectively in Tables D.1B and D.2B of Appendix D. The rising and recession curves with probabilities of 0.01, 0.02, 0.1, 0.5, 0.9 and 0.99 are given in Figs 5.6A and 5.6B.

4.6 Expected Hydrographs of Different Probable Floods

A frequency analysis was made on annual flood peaks and shown in Fig 5.7. The full hydrograph could be sketched for the given probable flood peak by choosing the same probable rising curve and as reasoned earlier the median recession curve.

CHAPTER V

RESULTS AND DISCUSSION

5.1 General

In this chapter the results of the analysis carried out are presented and discussed. The summary of frequency curves of the Q_{ratio} and Q_{mean} are presented; the median rising and recession curves are given for different periods of record and locations of river gauging stations. The computed flood hydrographs are compared with those of historical floods of corresponding frequencies. The effect of the record length is also discussed.

5.2 Frequency Analysis of Q_{ratio}

The frequency histograms of Q_{ratio} presented in Appendix A are summarized as frequency distribution curves in Figs. 5.1A and 5.1B. These curves were obtained by approximating the histograms. For both rising and recession cases the distribution is skewed. The recession frequency curves are skewed to the right; the rising frequency curves are skewed to the left; the skewness decreases with discharge levels in both the cases.

Table V.1 shows the result of an analysis to obtain the skewness coefficients for the distributions. It appears that a correlation exists between the skewness coefficient and the discharge level. A correlation analysis indicates that these coefficients are 0.925 and -0.794 respectively for rising and recession cases. The frequency distribution curves (Figs. 5.1A and 5.1B) also show that skewness and discharge level are correlated. The probability

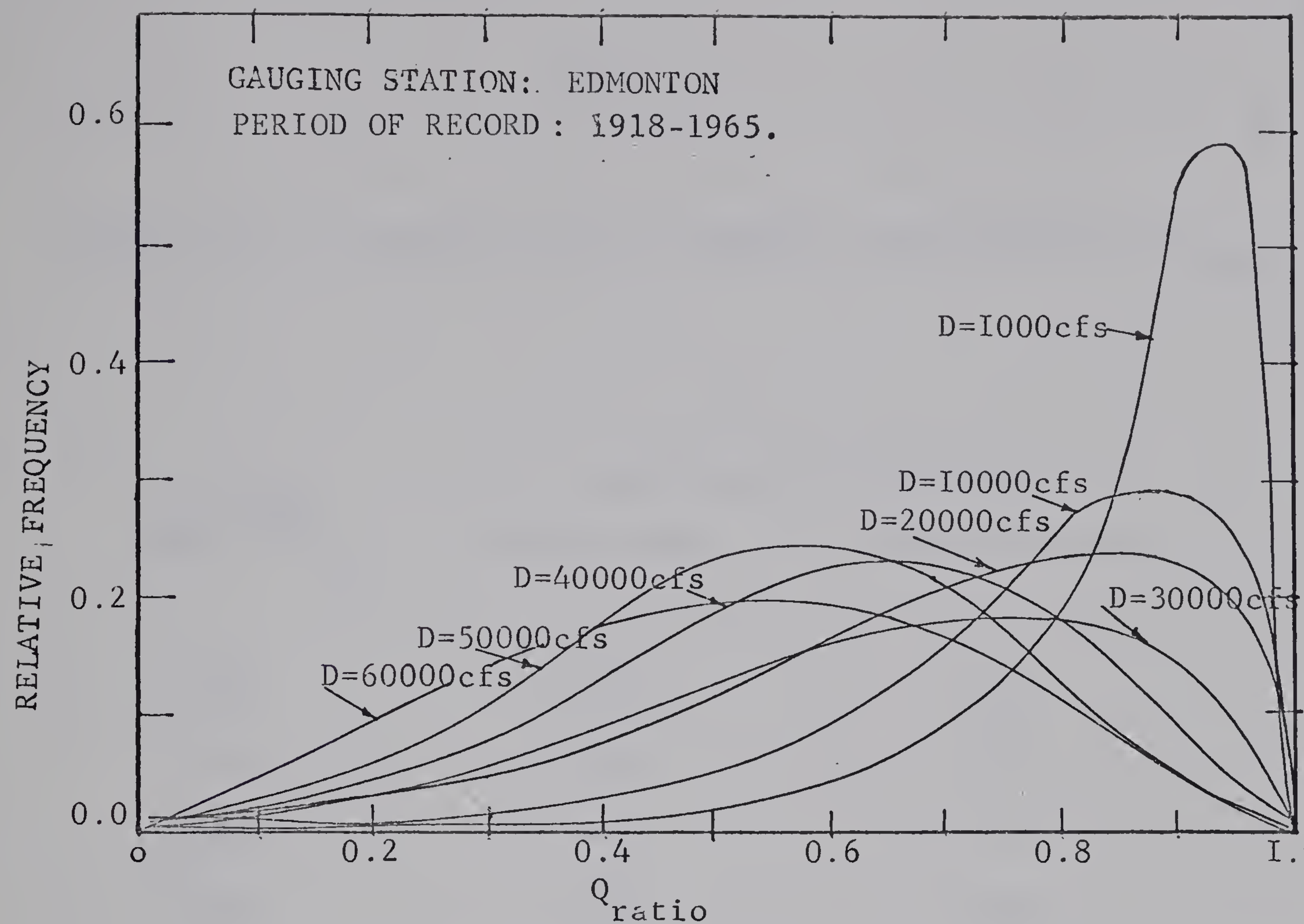


FIG. 5.1A FREQUENCY DISTRIBUTION CURVES OF Q_{RATIO}: RISING CASES.

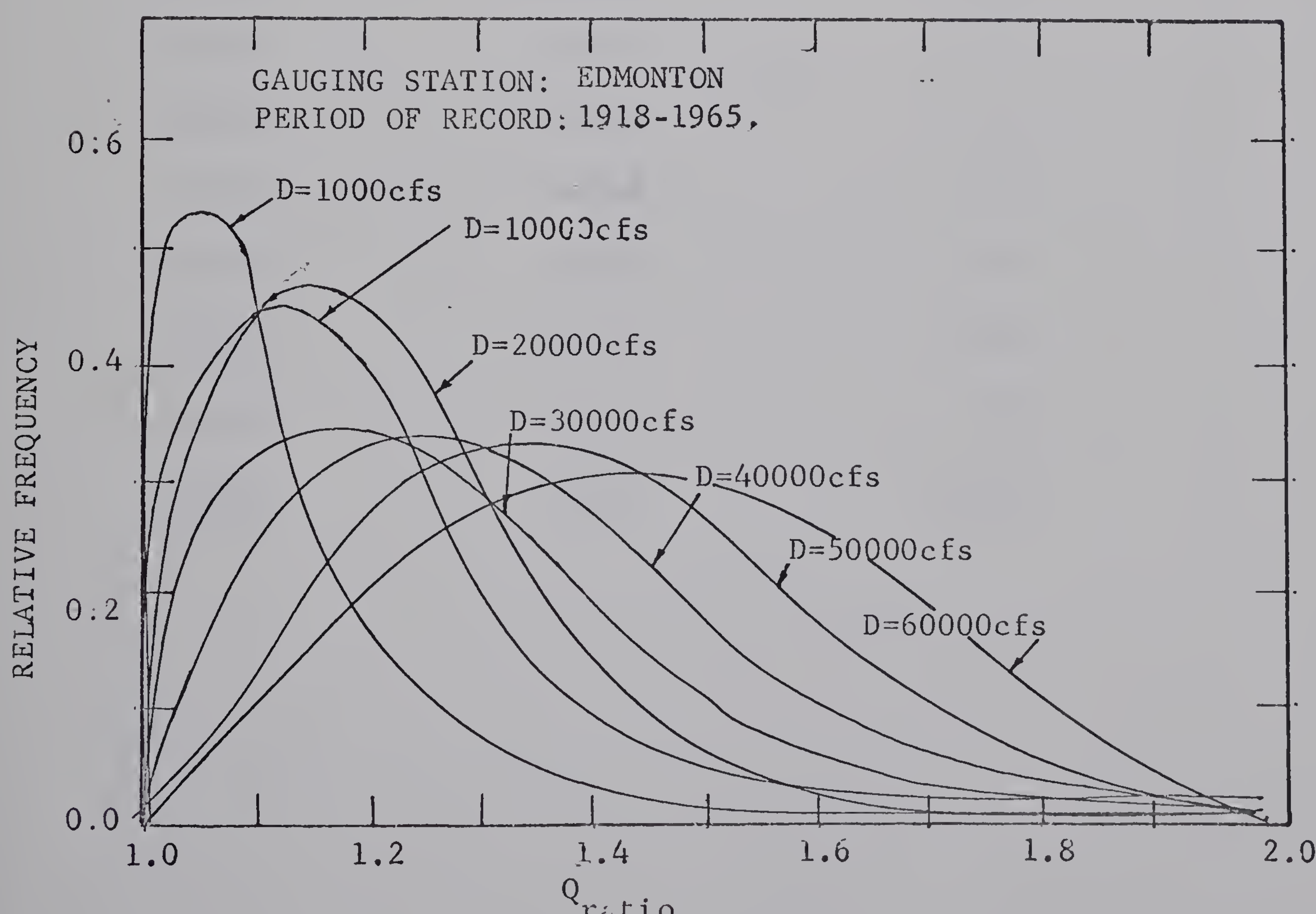


FIG. 5.1B FREQUENCY DISTRIBUTION CURVES OF Q_{RATIO}: RECESSION CASES.

TABLE V.1

COEFFICIENT OF SKEWNESS FOR THE FREQUENCY DISTRIBUTION OF Q_{ratio}

Discharge Level (cfs)	Coefficient of Skewness	
	Rising cases	Recession cases
1000	-1.874	2.195
5000	-1.287	3.206
10000	-1.287	4.531
15000	-1.220	3.254
20000	-0.854	2.506
25000	-0.715	2.283
30000	-0.265	2.009
35000	0.164	1.657
40000	0.052	1.155
45000	0.018	0.520
50000	-0.103	0.200
55000	0.412	0.423
60000	0.841	-0.038

curves (i.e. the cumulative frequency curves) of Q_{ratio} are given in Appendix C and are summarized in Figs. 5.2A and 5.2B.

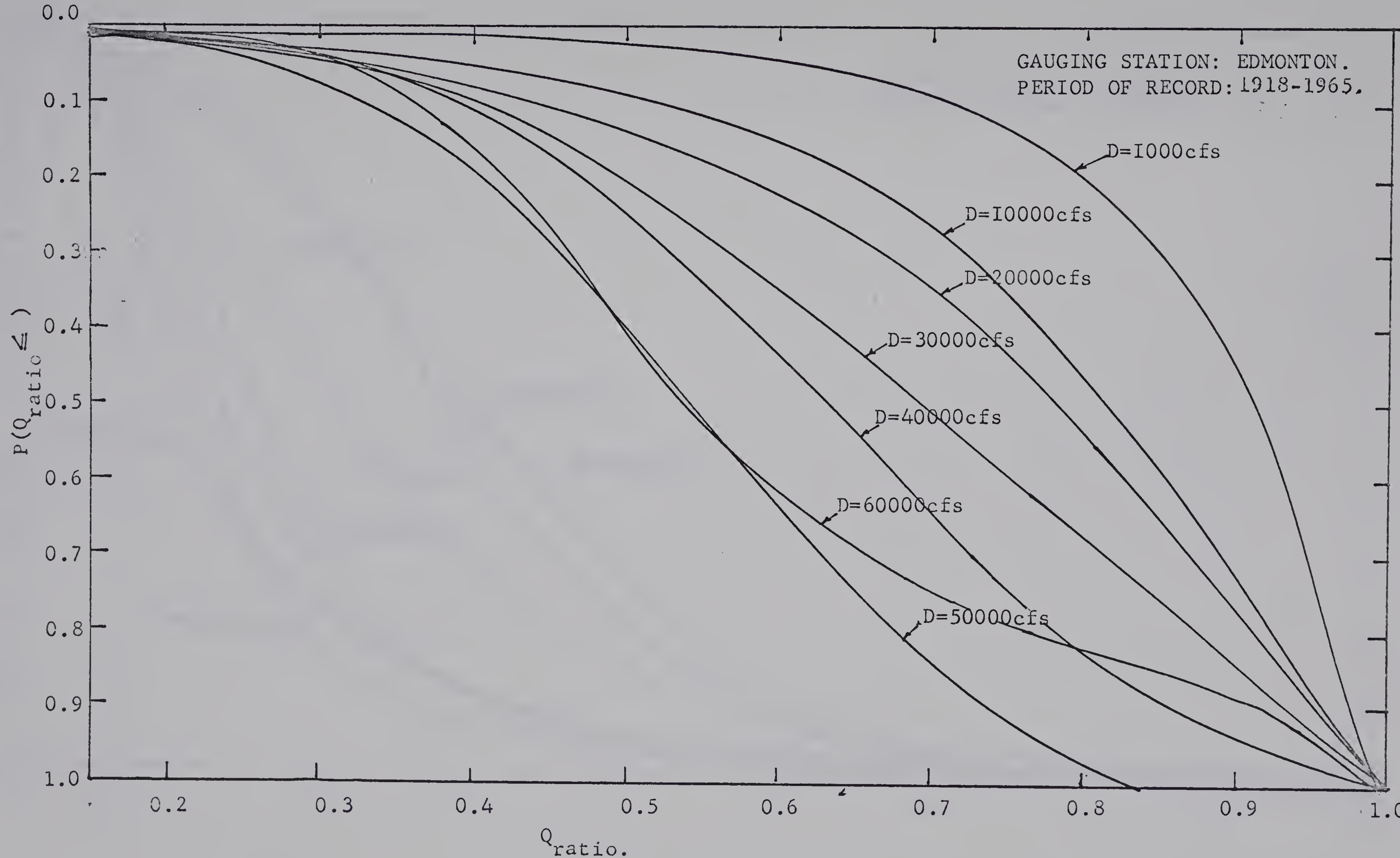
5.3 Frequency Analysis of Q_{mean}

In order to test the assumption that Q_1 and Q_2 are equally below and above D (and vice versa in the recession cases), a frequency analysis of Q_{mean} and a correlation analysis between Q_{ratio} and Q_{mean} were made. The frequency histograms of the standard variate $(Q_{mean}-D)/S$ are given in Appendix B.

The sample points were plotted on a log-normal paper and the results shown in Figs. 5.3A and 5.3B clearly indicate that the distribution is approximately log-normal. The correlation coefficients between the Q_{ratio} and Q_{mean} are given in Table V.2. These poor correlations show the independency of the two variables.

5.4 Median Rising and Recession Curves

As stated earlier in Chapter IV, the various probable curves were developed. They are shown in Figs. 5.6A and 5.6B. In this section, only the median (0.5 probability) curves are considered and are shown in Figs. 5.4A, 5.4B, 5.5A and 5.5B. The individual points are less deviated from the median curve in the recession case than in the rising case, showing the consistency in the recession characteristics. The larger deviations in the rising case may be attributed to the varying climatic characteristics that affect the precipitation over the catchment and catchment state. Figs. 5.4A and 5.4B show the median curves for the flows of different record periods at Edmonton. These



PROBABILITY CURVES OF Q_{RATIO} : RISING CASES.

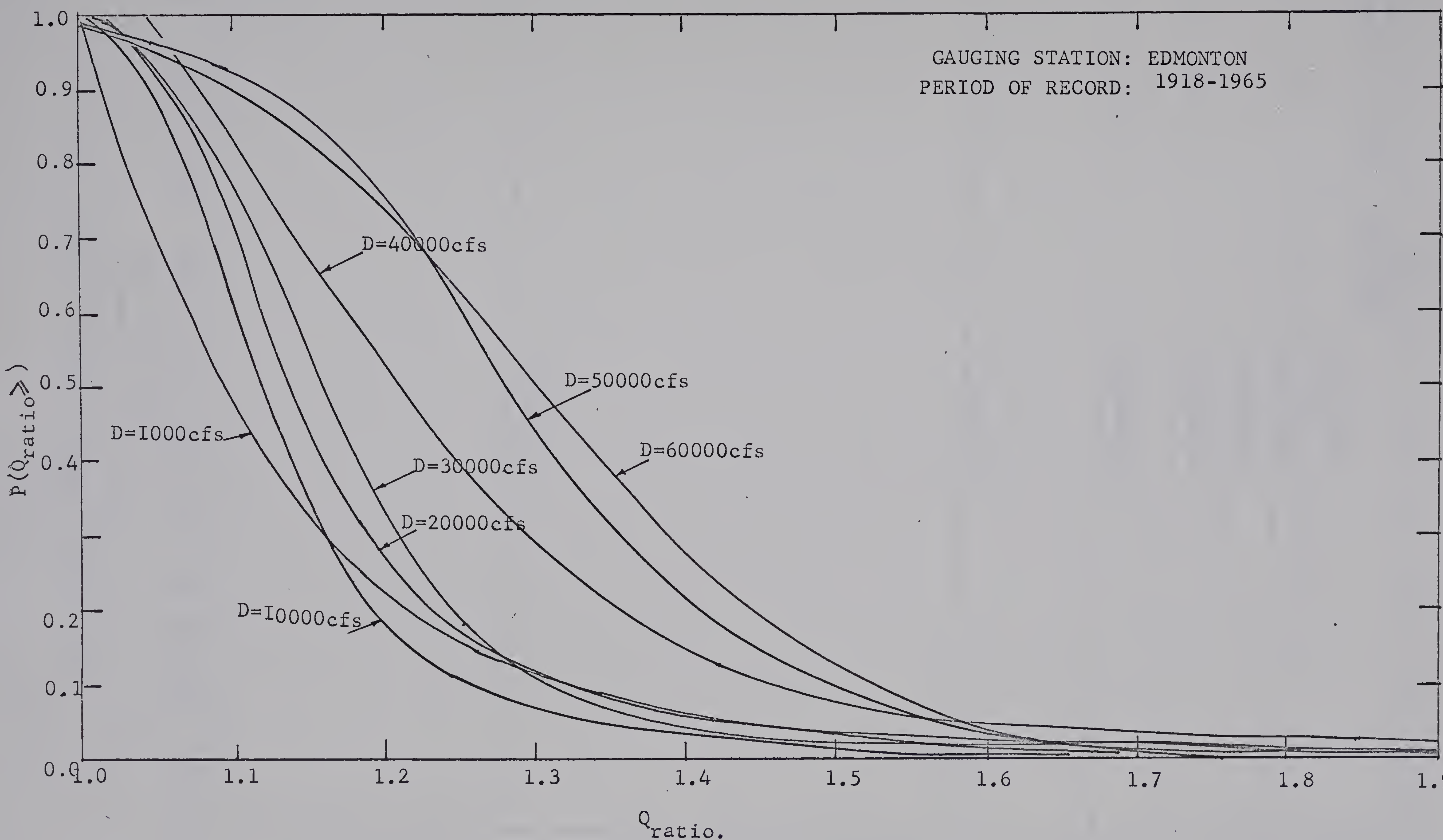


FIG. 5.2B PROBABILITY CURVES OF Q_{RATIO}: RECESSION CASES.

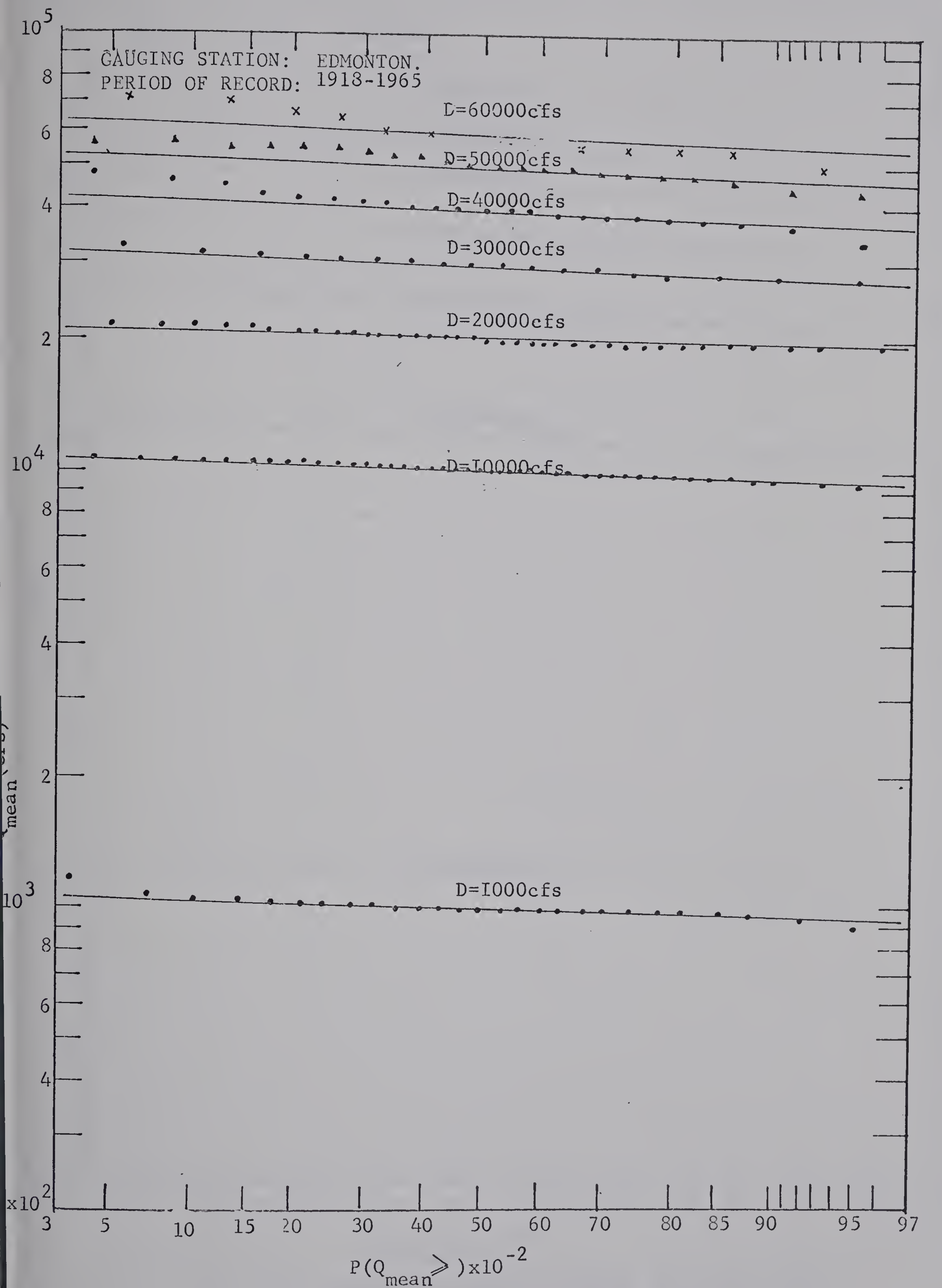


FIG. 5.3A FREQUENCY CURVES OF Q_{MEAN}: RISING CASES.

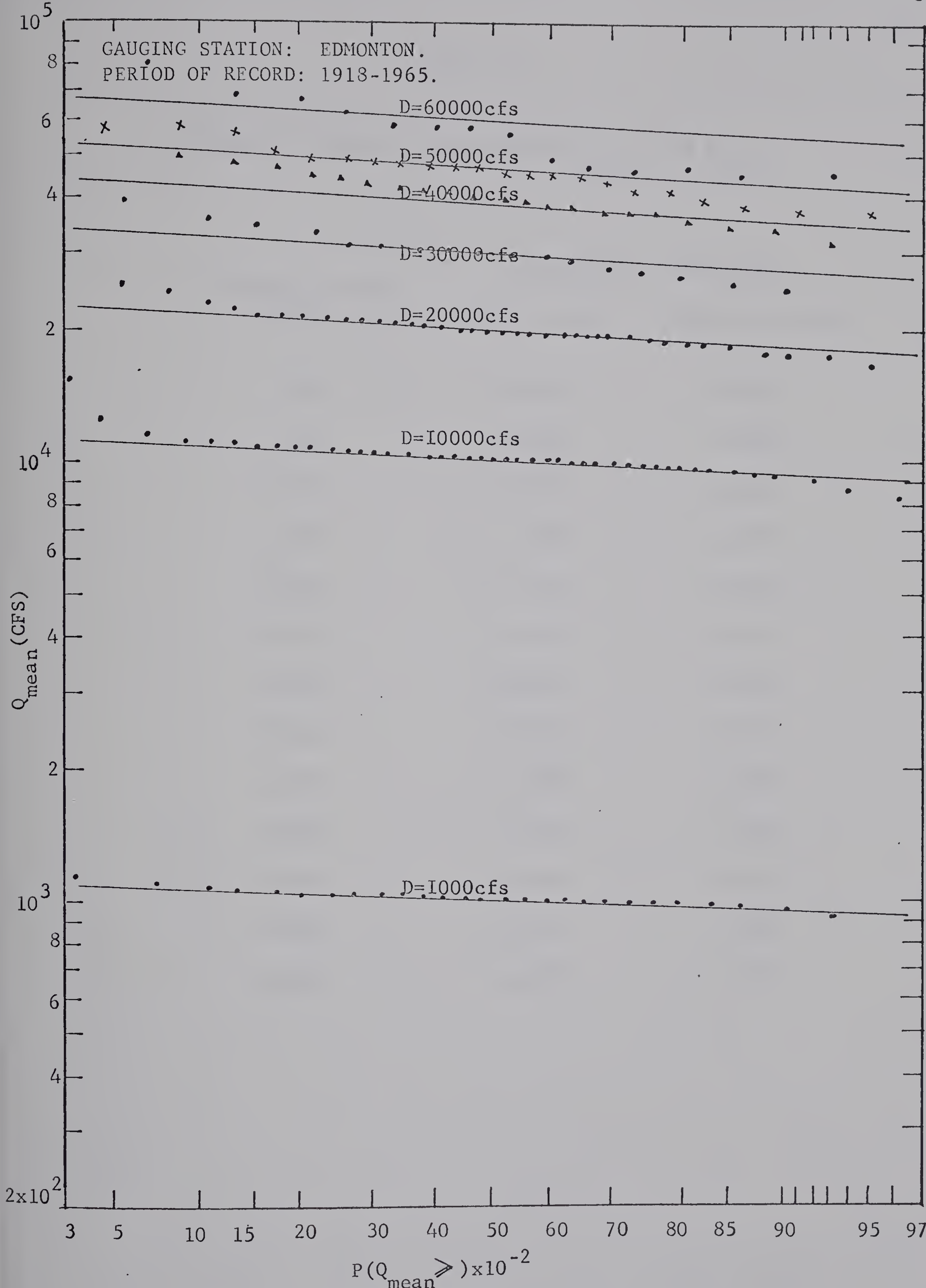


FIG. 5.3B FREQUENCY CURVES OF Q_{MEAN}: RECESSION CASES.

TABLE V.2

CORRELATION COEFFICIENTS BETWEEN Q_{ratio} AND Q_{mean}

Discharge Level (cfs)	Correlation Coefficient	
	Rising cases	Recession cases
1000	-0.3445	0.2828
5000	-0.6039	0.3662
10000	-0.3365	-0.0413
15000	-0.4009	0.0244
20000	-0.3033	-0.0593
25000	-0.2672	0.2646
30000	-0.0295	0.0688
35000	-0.0156	0.2586
40000	-0.1328	0.0492
45000	-0.0089	0.3305
50000	0.2663	0.0402
55000	0.2734	0.5369
60000	0.3098	0.0074

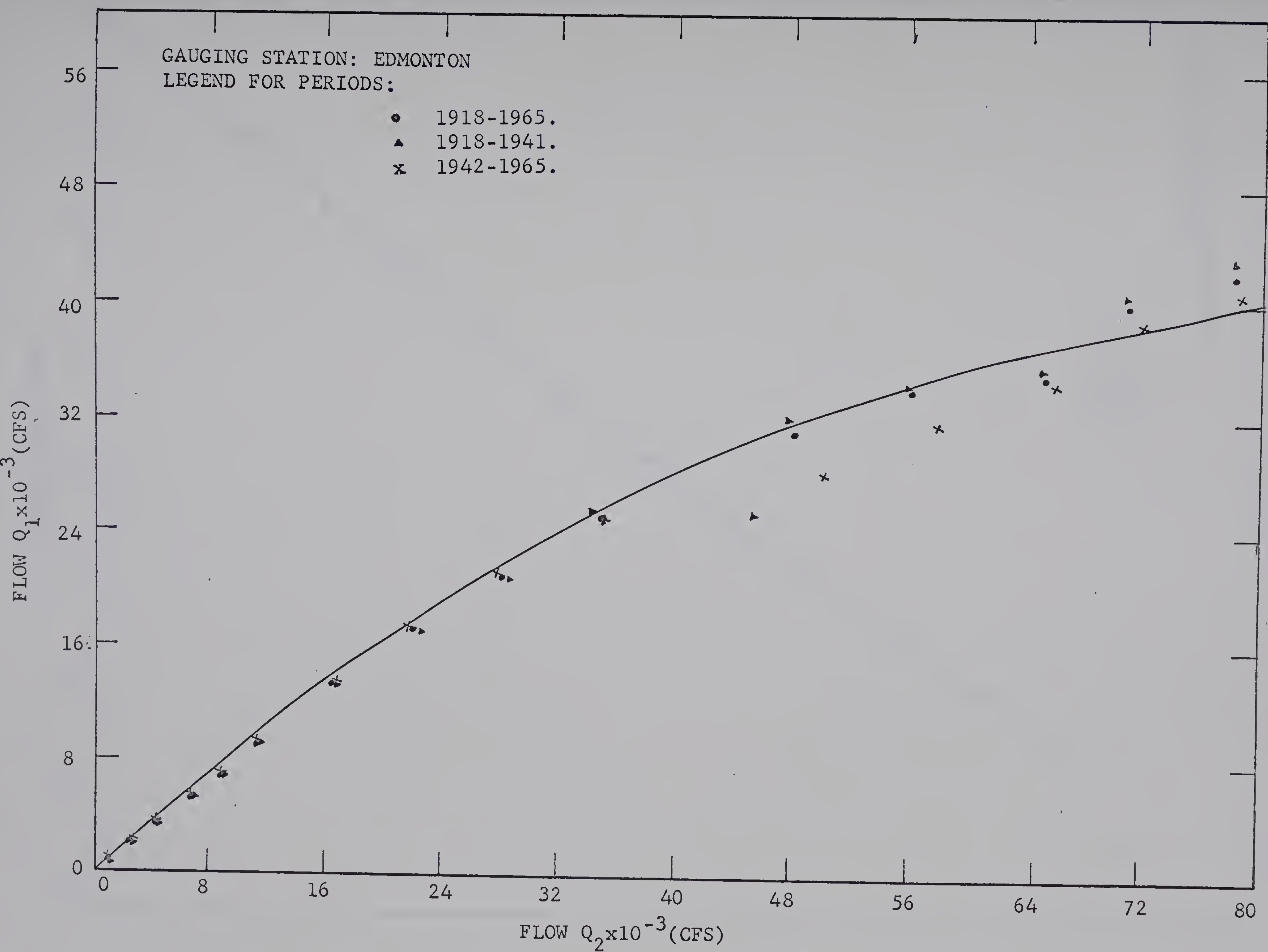


FIG. 5.4A MEDIAN RISING CURVES FOR DIFFERENT RECORD PERIODS.

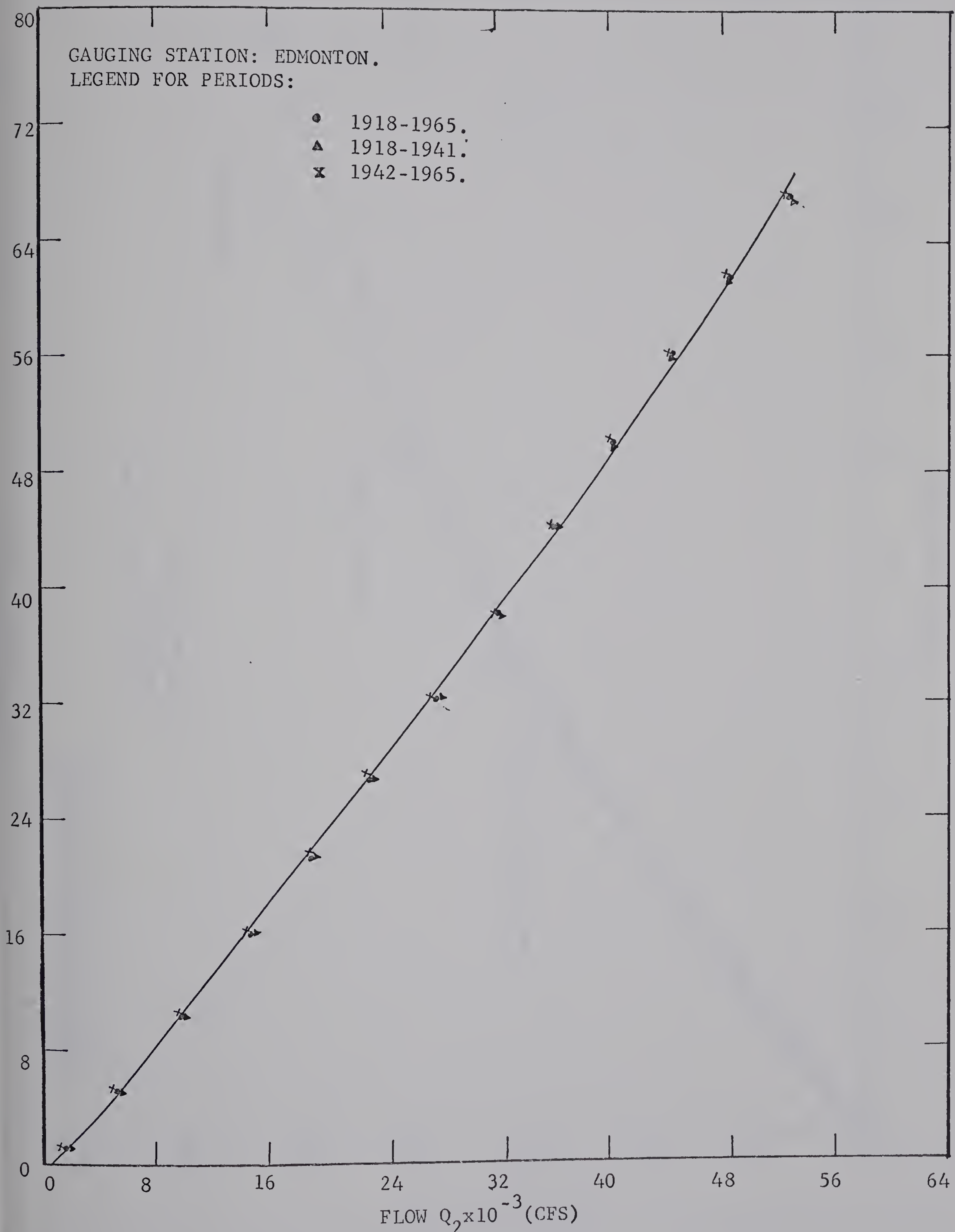


FIG. 5.4B MEDIAN RECESSION CURVES FOR DIFFERENT RECORD PERIODS.

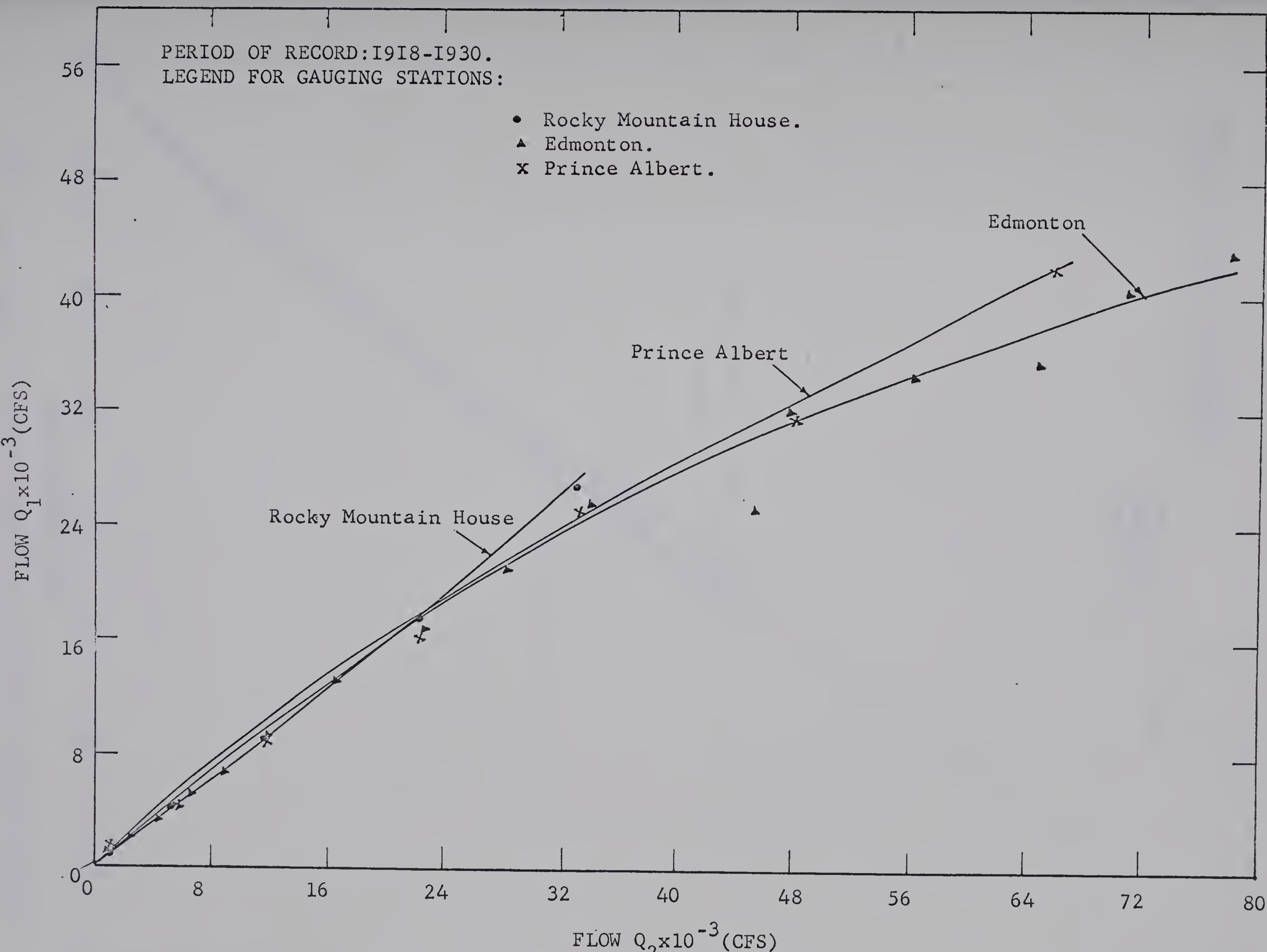


FIG. 5.5A MEDIAN RISING CURVES FOR DIFFERENT GAUGING STATIONS.

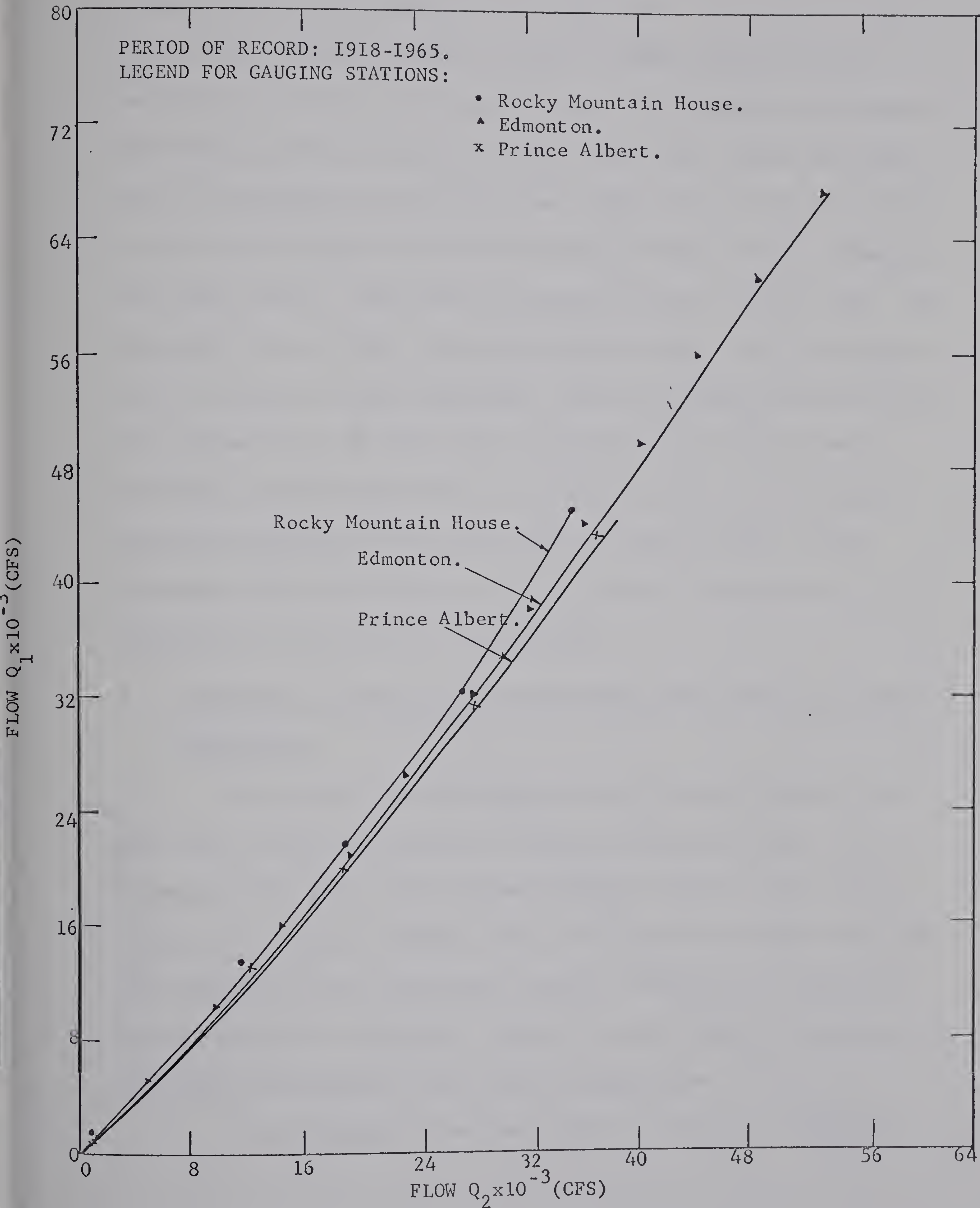


FIG. 5.5B MEDIAN RECESSION CURVES FOR DIFFERENT GAUGING STATIONS.

figures show the homogeneity of the population. As it will be concluded later, a record length of about 13 years was found to be sufficient to arrive at the median curves with reasonable percentage deviation from those computed from the full record length (48 years). For the overlapping record of the years 1918 through 1930, the median curves were developed for the flow at Rocky Mountain House, Edmonton, and Prince Albert. The curves are shown in Figs. 5.5A and 5.5B. The recession curves of Fig. 5.5B show somewhat slower rate of recession for the stations further downstream. From the rising curves no definite conclusion can be drawn, only the randomness of points can be observed, and this may be due to the random occurrence of precipitation and catchment state with respect to area and time. This emphasizes that the recession curve is a better characteristic to describe the basin than the rising curve.

5.5 Comparison of Expected Flood Hydrographs with Historical Flood Hydrographs

The various probable curves shown in Figs. 5.6A and 5.6B were used to obtain hydrographs of required probable floods. The necessary flood peaks were obtained from the frequency curve given in Fig. 5.7. The two straight lines fitting the flood peak date show the existence of two populations: possibly the snow melt flood and floods produced by rainstorms. However, in the present analysis the data have been assumed to come from one population.

The hydrographs developed using the rising and recession

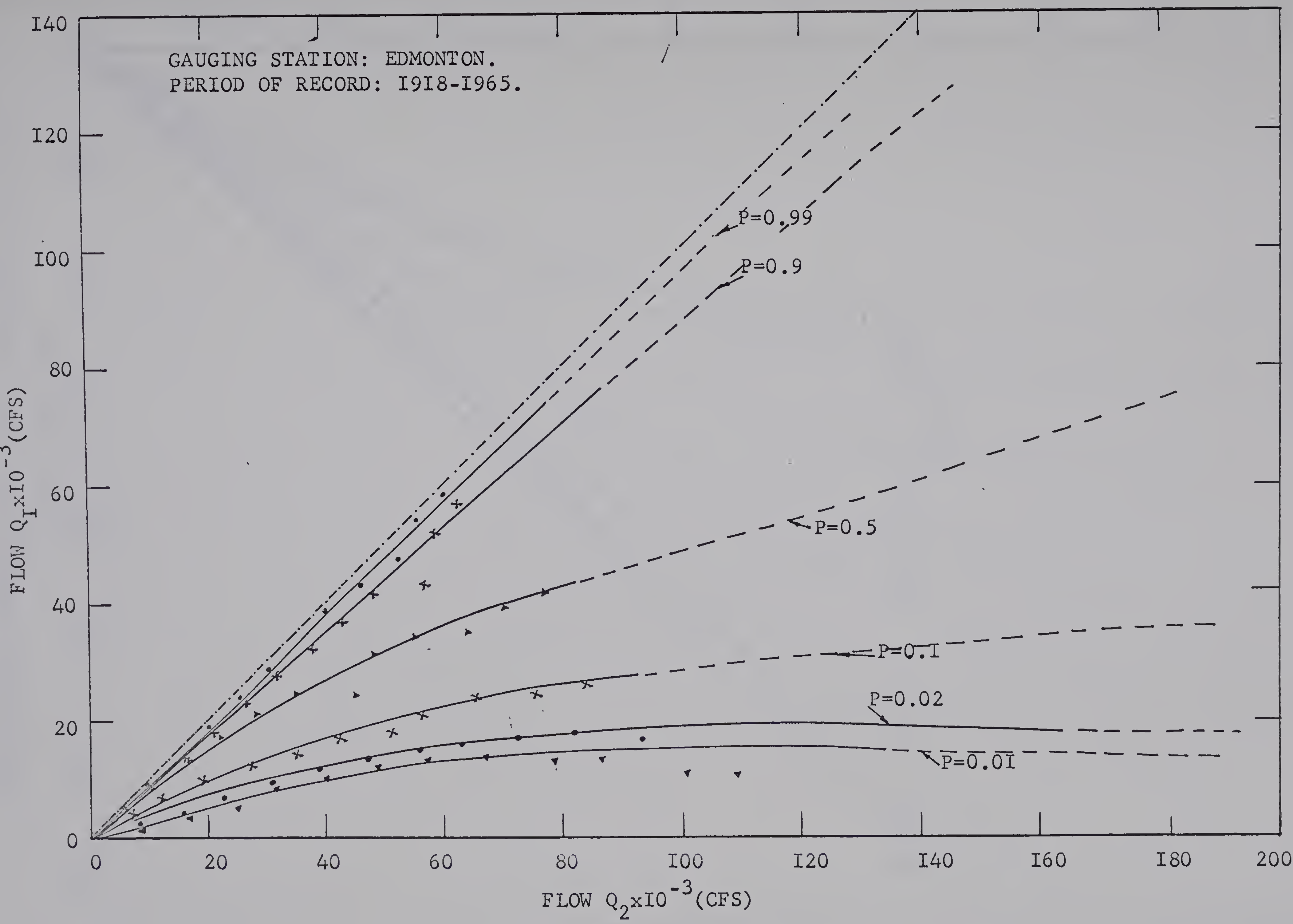


FIG. 5.6A RISING CURVES OF DIFFERENT PROBABILITIES.

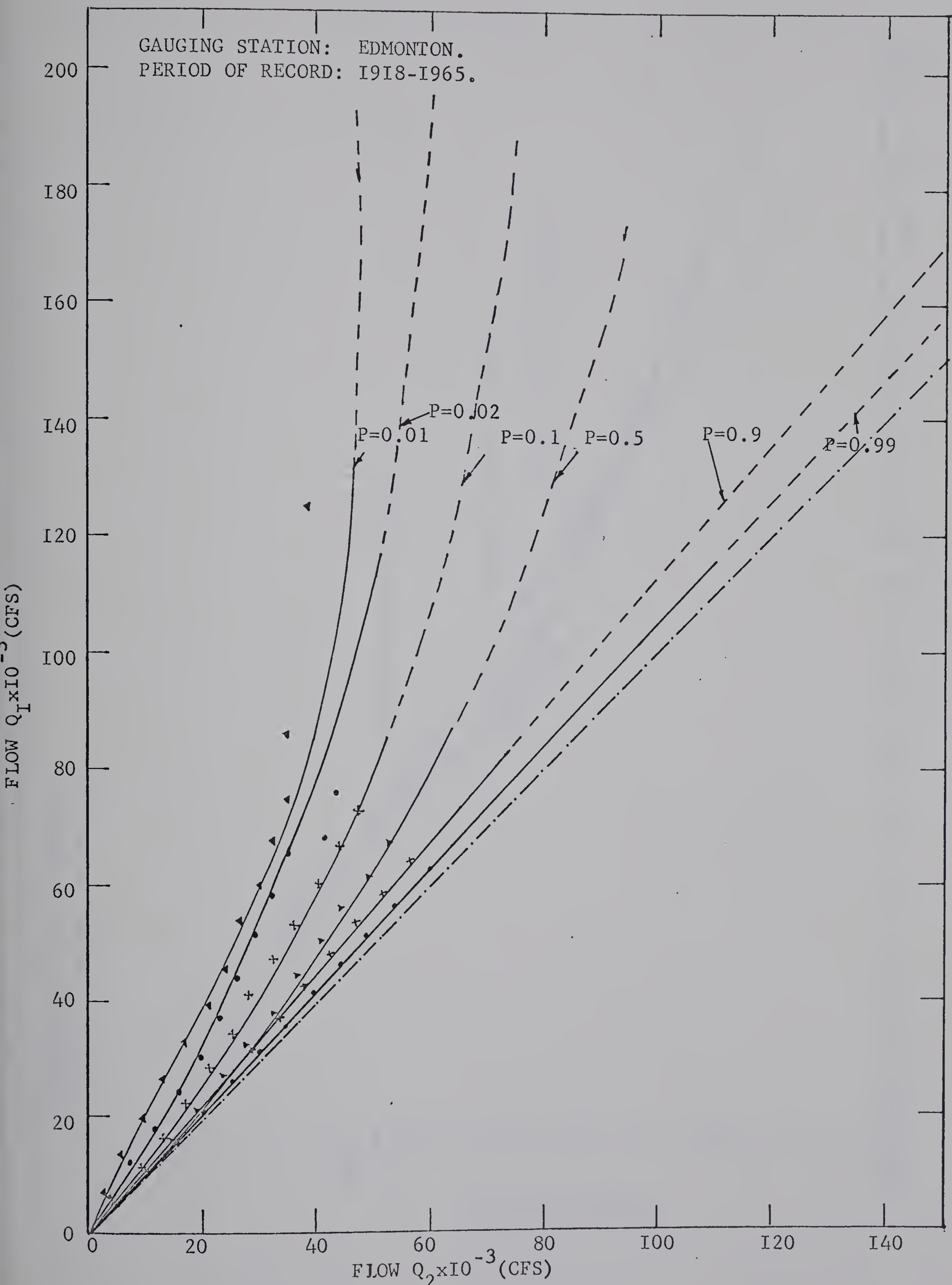


FIG. 5.6B RECESSION CURVES OF DIFFERENT PROBABILITIES.

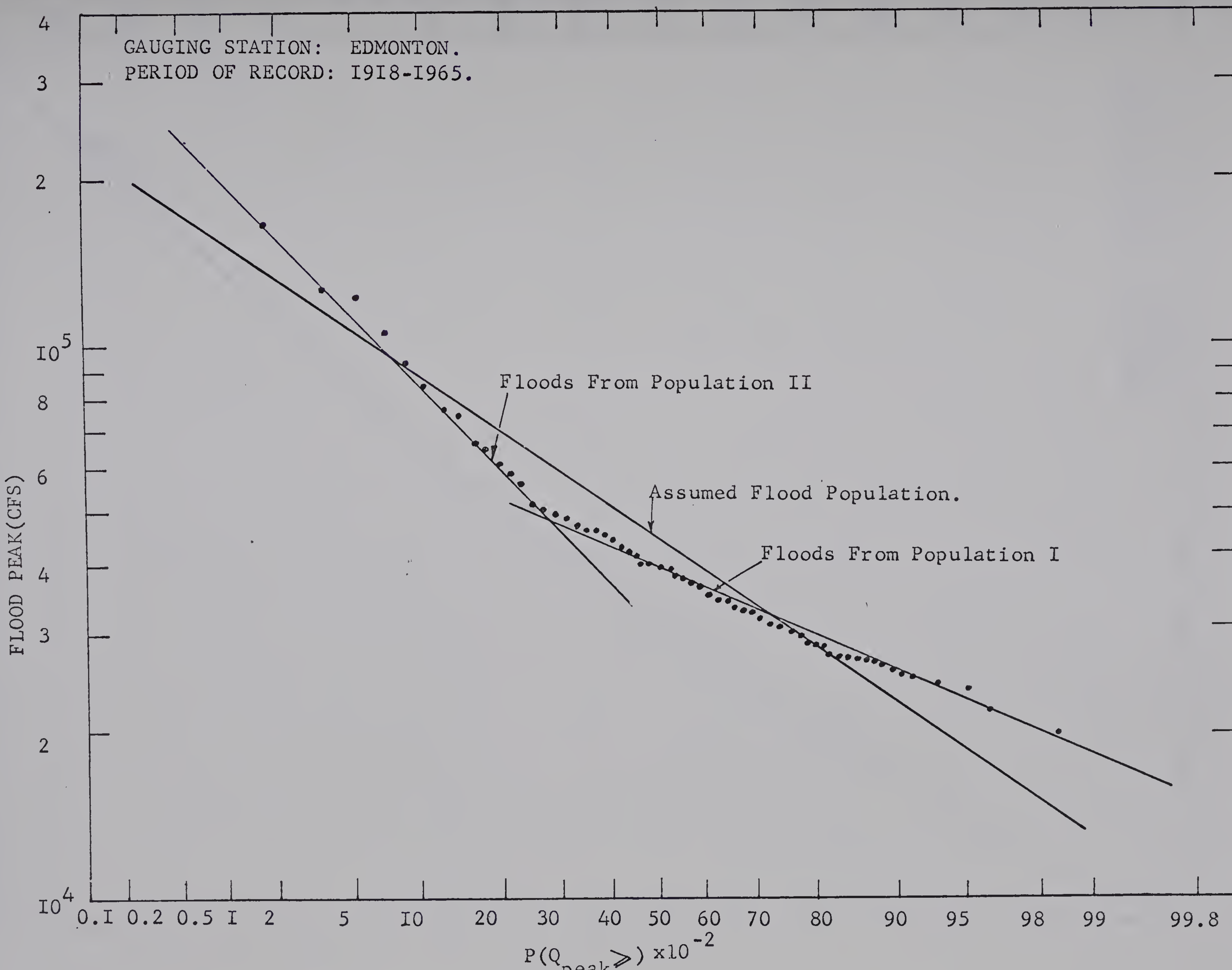


FIG. 5.7 ANNUAL FLOOD PEAK FREQUENCY CURVE

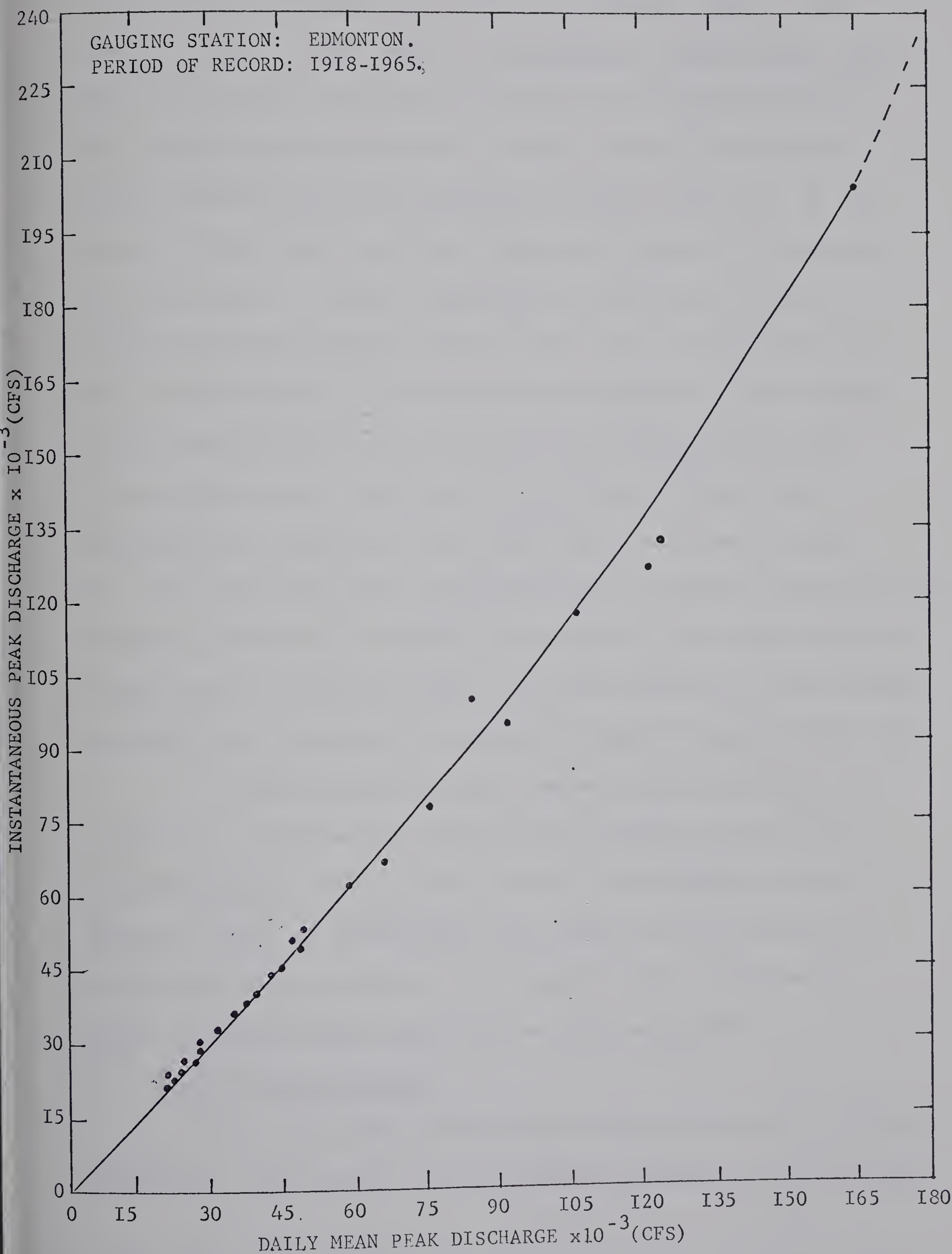


FIG. 5.8 DAILY MEAN PEAK vs INSTANTANEOUS PEAK CURVE.

curves should be compared with those of historical floods of the same frequency. In this study the hydrograph of 0.02 probable flood that has occurred in 1952 was plotted and the expected hydrograph that could be developed with 0.02 probable rising curve and the median recession curve was superposed as shown in Fig 5.9. In the figure are also shown the median rising curve and the 0.02 probable recession curve. A similar comparison has been made for the 0.1 probable flood-hydrograph as given in Fig 5.10. In both the cases the recession portion of the expected hydrographs fits more closely to the same portion of the historical flood hydrograph than the rising portion does. The computed flow volumes obtained from hydrographs of expected and historical floods are given in Table V.3. This table shows the percent deviation of expected volume given by different combinations of rising and recession curves from the actual volume produced during the flood. For the computation of the volumes arbitrary base flow lines were assumed as shown in Figs 5.9 and 5.10.

In the foregoing analysis the mean daily peak value was considered. But often the instantaneous peak must be used in the flood hydrograph. From the flow records of the gauging station at Edmonton a curve of instantaneous peak against daily mean peak was obtained and given in Fig 5.8. This type of curve can be used to deduce the instantaneous peak from the daily mean peak.

5.6 Effect of Record Length

It is well known that more and better data result in better prediction. Unfortunately, often important decisions have to be made

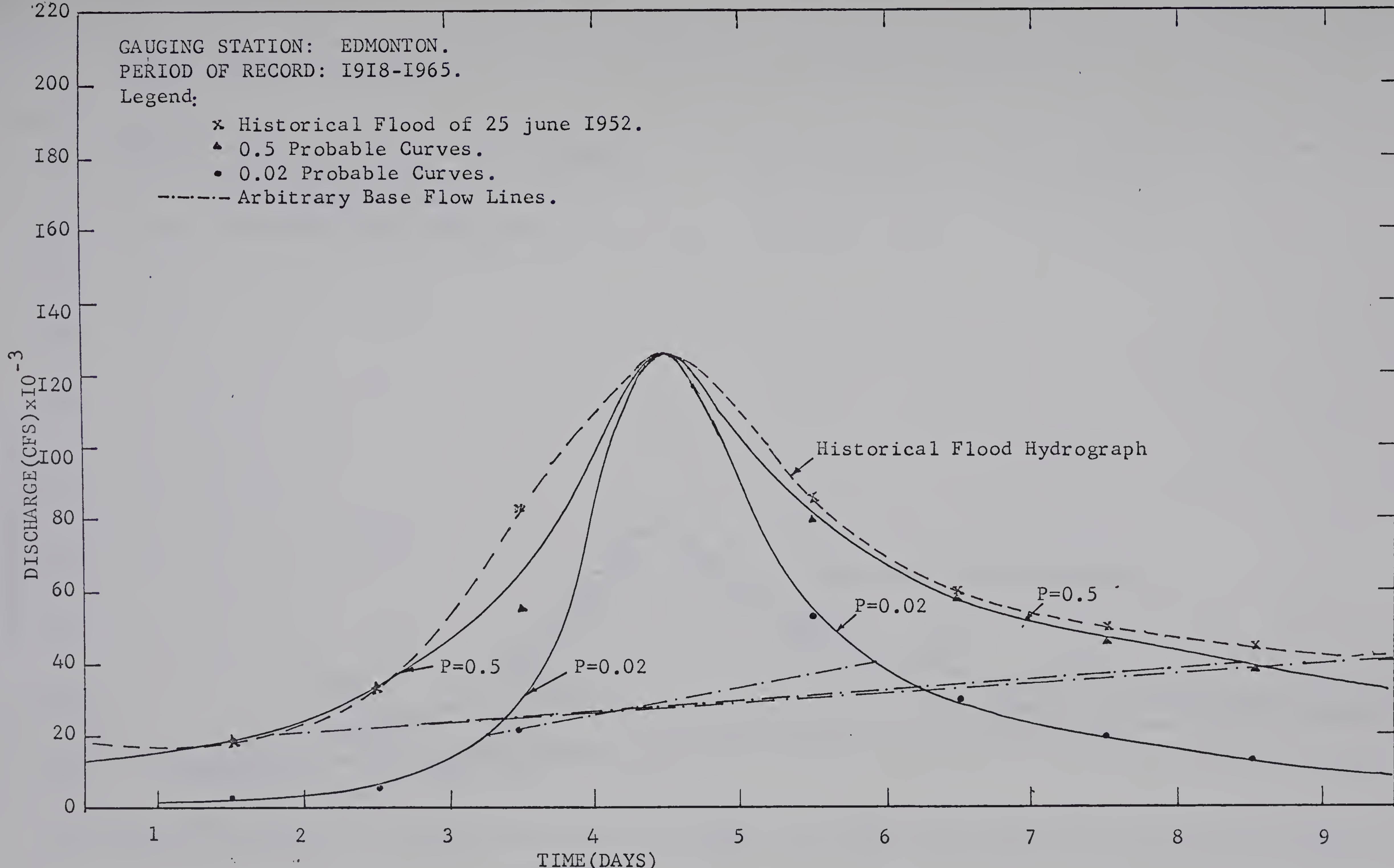


FIG. 5.9 HYDROGRAPH OF 0.02 PROBABLE FLOOD.

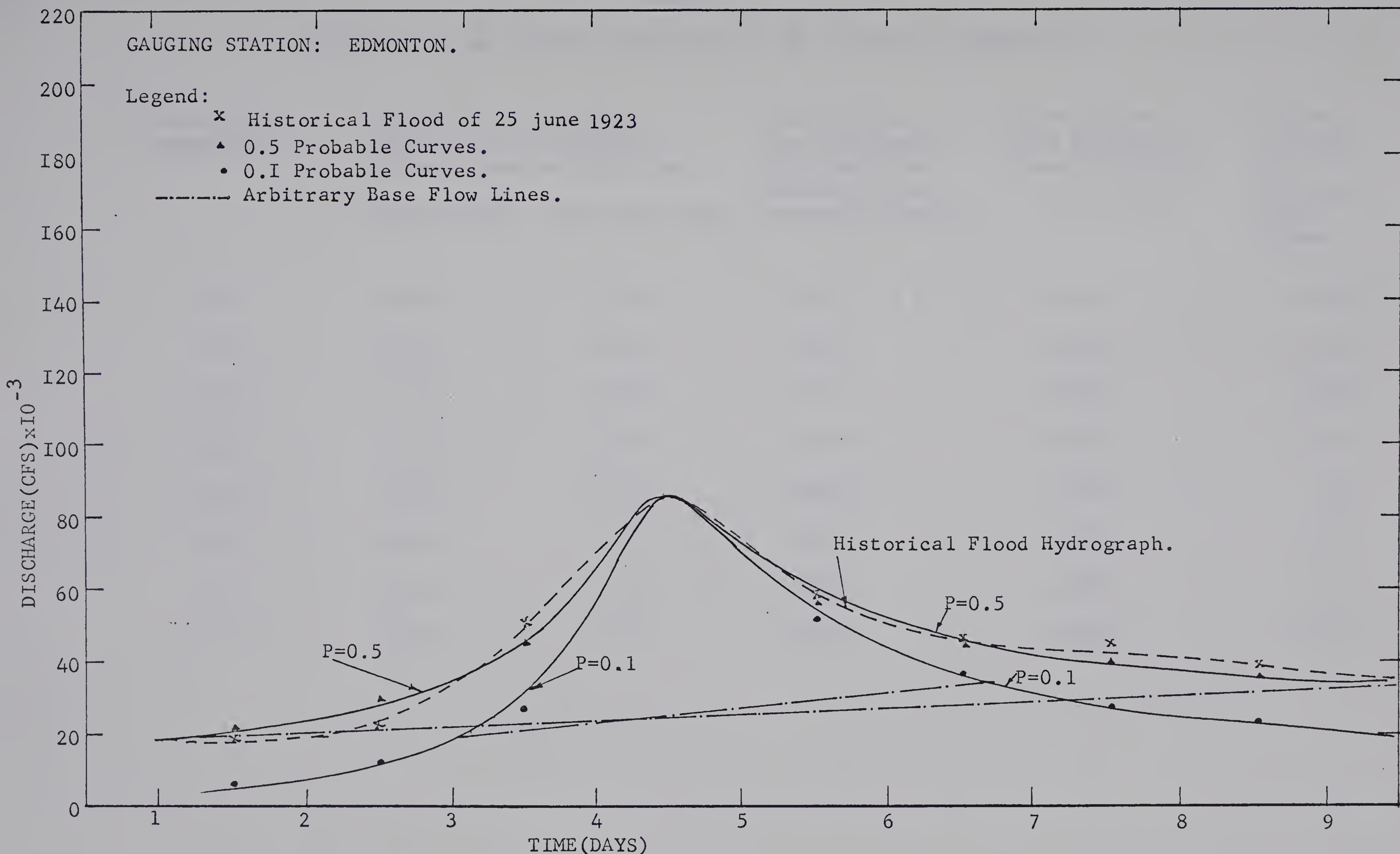


FIG. 5.I0 HYDROGRAPH OF vs PROBABLE FLOOD.

TABLE V.3

COMPARITION OF EXPECTED HYDROGRAPHS WITH HISTORICAL HYDROGRAPHS

Flood Peak Probability	Probability of Curves Composing the Hydrograph of the Probable Flood Peak: Rising Curve	Recession Curve	Flood Volume from Hydrograph with the given Combination of Curves (cfs.days)	Flood Volume from Historical Flood Hydrograph (cfs. days)	Percent Deviation from Historical Flood Volume
0.02	0.50	0.50	268500	300000	-10.30
0.02	0.50	0.02	198000	300000	-34.00
0.02	0.02	0.50	216000	300000	-28.00
0.02	0.02	0.02	146000	300000	-51.40
0.10	0.50	0.50	168000	155000	8.40
0.10	0.50	0.10	131000	155000	-15.50
0.10	0.10	0.50	144000	155000	- 7.10
0.10	0.10	0.10	107000	155000	-31.00

in spite of hydrologic deficiencies. The determination of the design flood hydrograph is not an exception to this. In this section the median rising and recession curves, for the gauging station at Edmonton computed from shorter periods of records are compared with those computed from full record length (48 years) considered. This comparison was made by computing the percent maximum of the deviations of the Q_{ratio} for shorter record length from those computed for full record length. Samples from successive years of 10, 15, 20, ..., 45, 48 were taken from the 48 year series. For example, the first sample using a record length of 10 years included the years 1 to 10 of the period of record, second sample of 10 years contained the events in the years from 2 to 11 years and so on. The last sample contained the events that occurred in the years from 39 to 48 in the sequence. In this way a series of 48 years yielded 39 samples of 10 years récord length. By this procedure the number of samples that could be formed with different record length is given by

$$n = M - N + 1 \qquad 5.1$$

where

n = the number of the samples.

M = the total of years in the series from which the samples were taken.

N = the total number of years in which the events were considered for sampling.

Thus the number of the samples for record length of 10, 15, 20, ..., 45

and 48 that could be formed from a series of 48 years is 39, 34, 29 ... 4 and 1. As the statistical measure, the median of Q_{ratio} for all the discharge levels considered was computed from each of these samples. Thus, the 39 samples containing events of 10 year period yielded 39 values of Q_{ratio}-medians for each D value. Thus, this is the analysis of all successive 10 year periods contained in the 48 years of record. Taking the measures of the 48 years series as the base, the maximum of the deviations, both positive and negative, of the sample estimates are shown in Figs. 5.11. On these figures are shown the maximum of the deviations of Q_{ratio}-median of shorter record for both rising and recession cases from those obtained from a base record of 48 years. The computed results are given in Appendix E.

Envelopes have been drawn enclosing the data points. Figs. 5.11 show that the envelopes for lower discharge levels are regular and follow a mathematical equation of the form

$$y = \frac{C}{N} \tag{5.2}$$

where y is the percent deviation (either positive or negative) and C is a constant to be determined empirically. The greater scattering of points for higher discharge levels may be due to the comparatively smaller sample sizes at higher discharge levels. From the results in Figs. 5.11 it can be concluded that a record length of about 13 years would be sufficient for the present analysis. On the basis of this conclusion the analysis on the records of two other stations comprised the overlapping record of 13 years, (1918 through 1930).

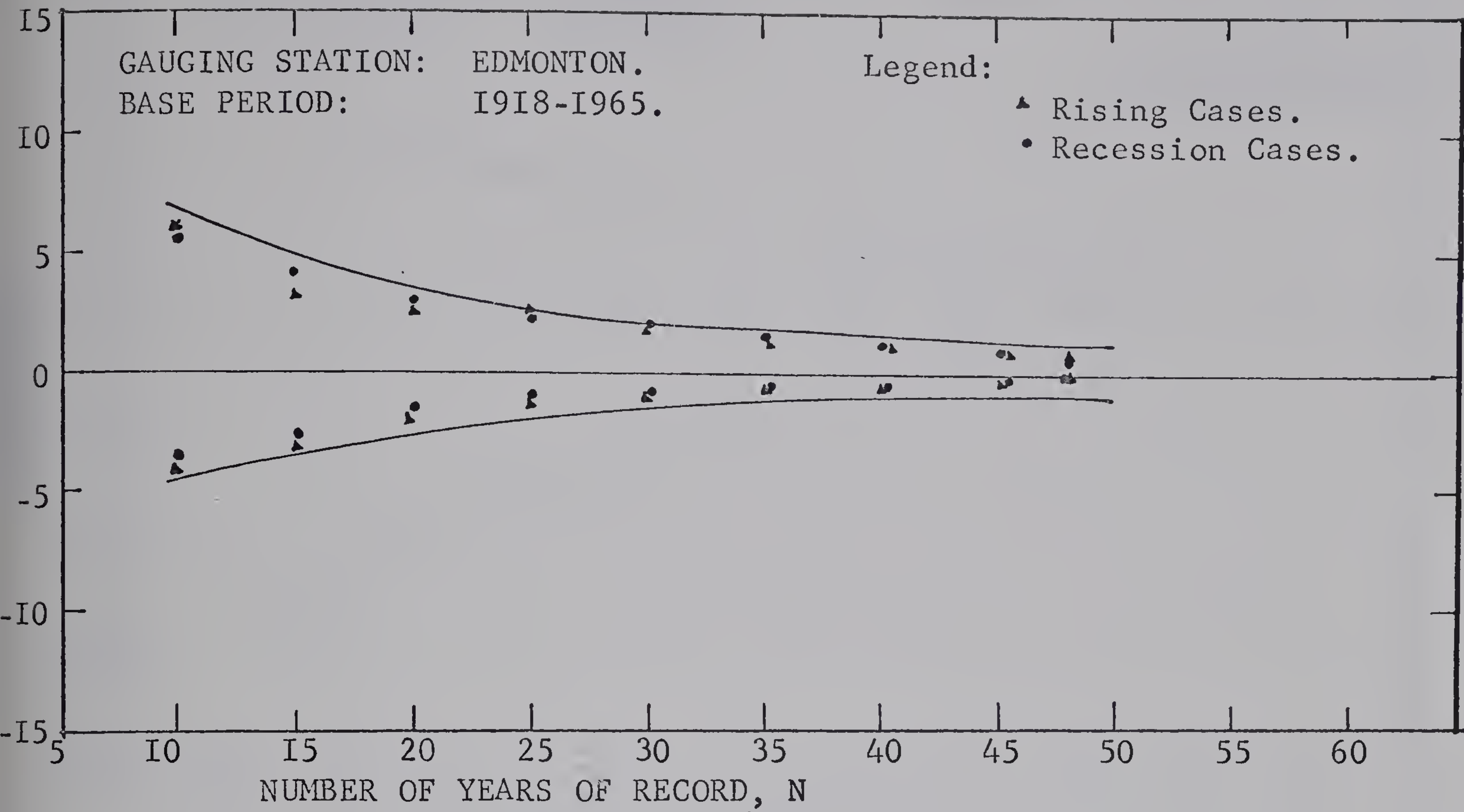


FIG. 5.11A MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=1000(CFS)

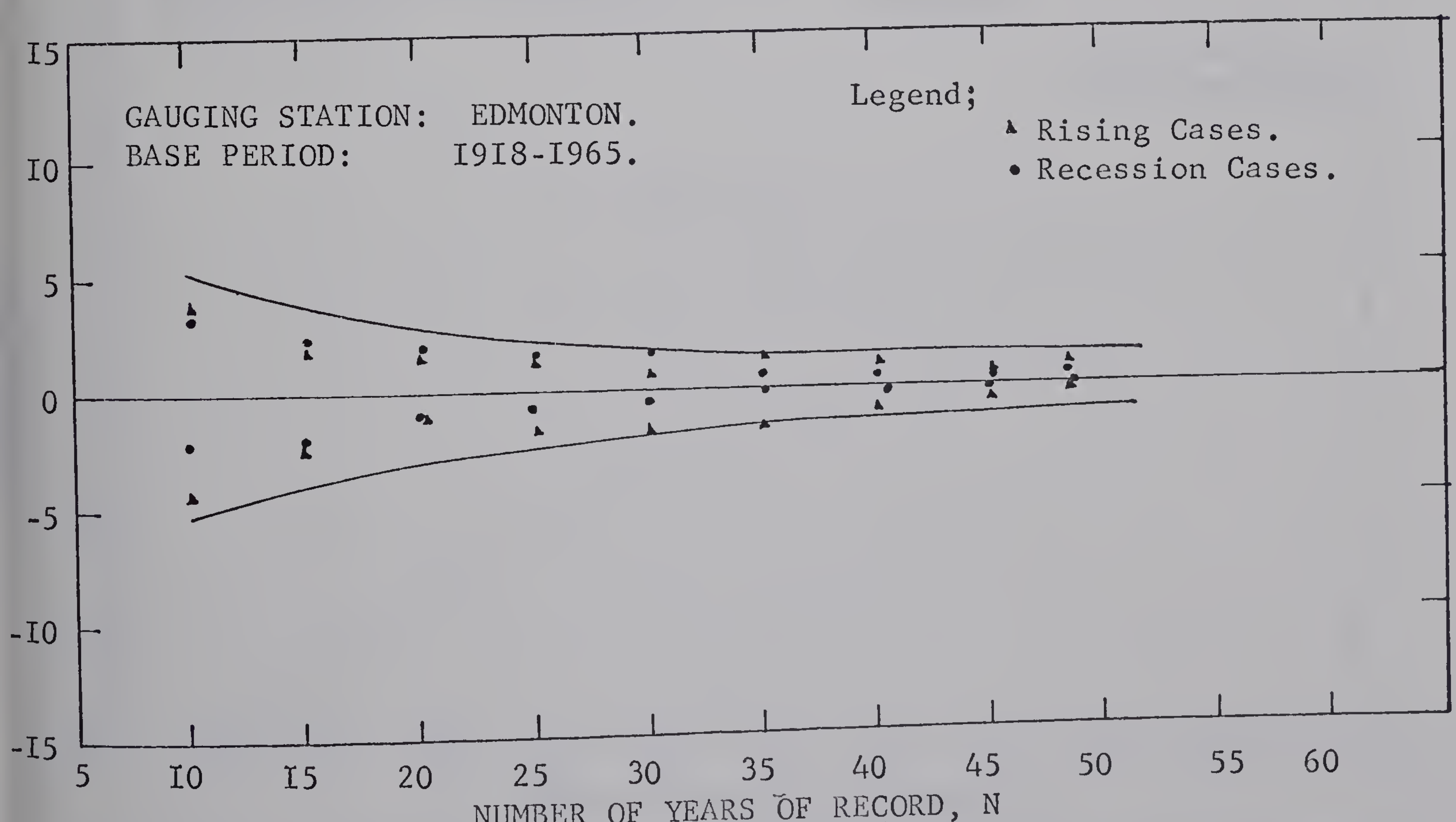


FIG. 5.11B MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=10000(CFS)

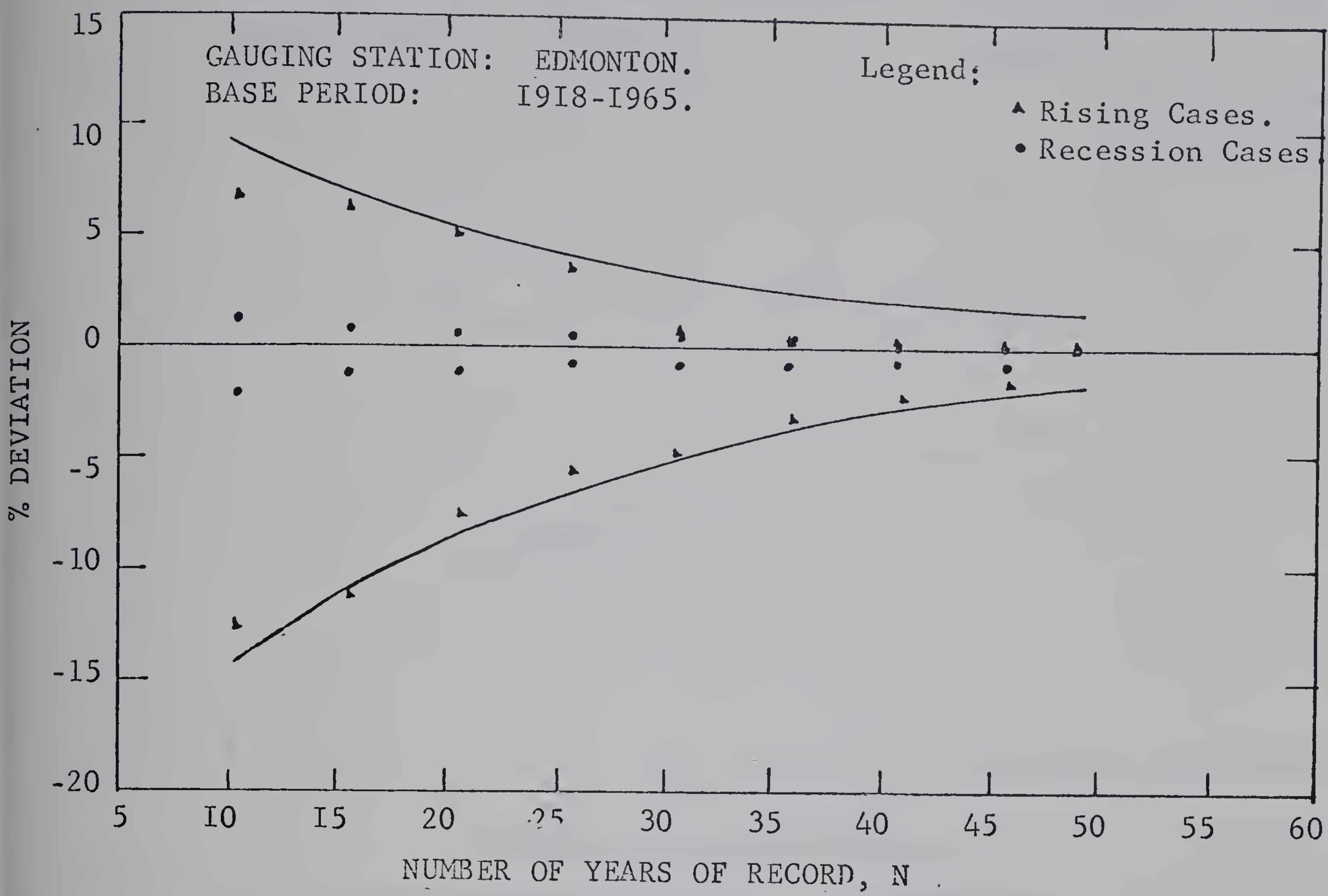


FIG. 5.11C MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=20000(CFS)

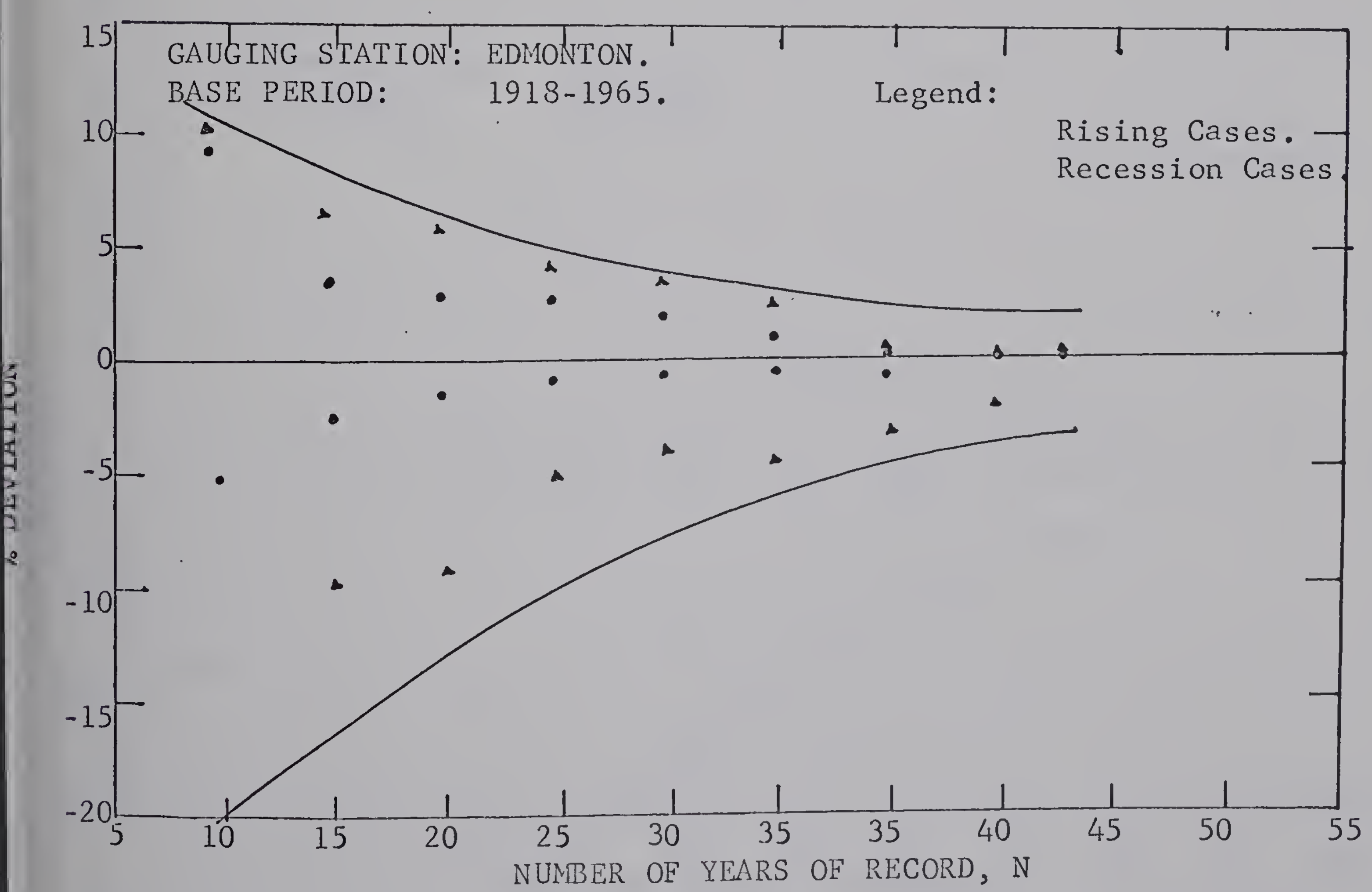


FIG. 5.11D MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=30000(CFS)

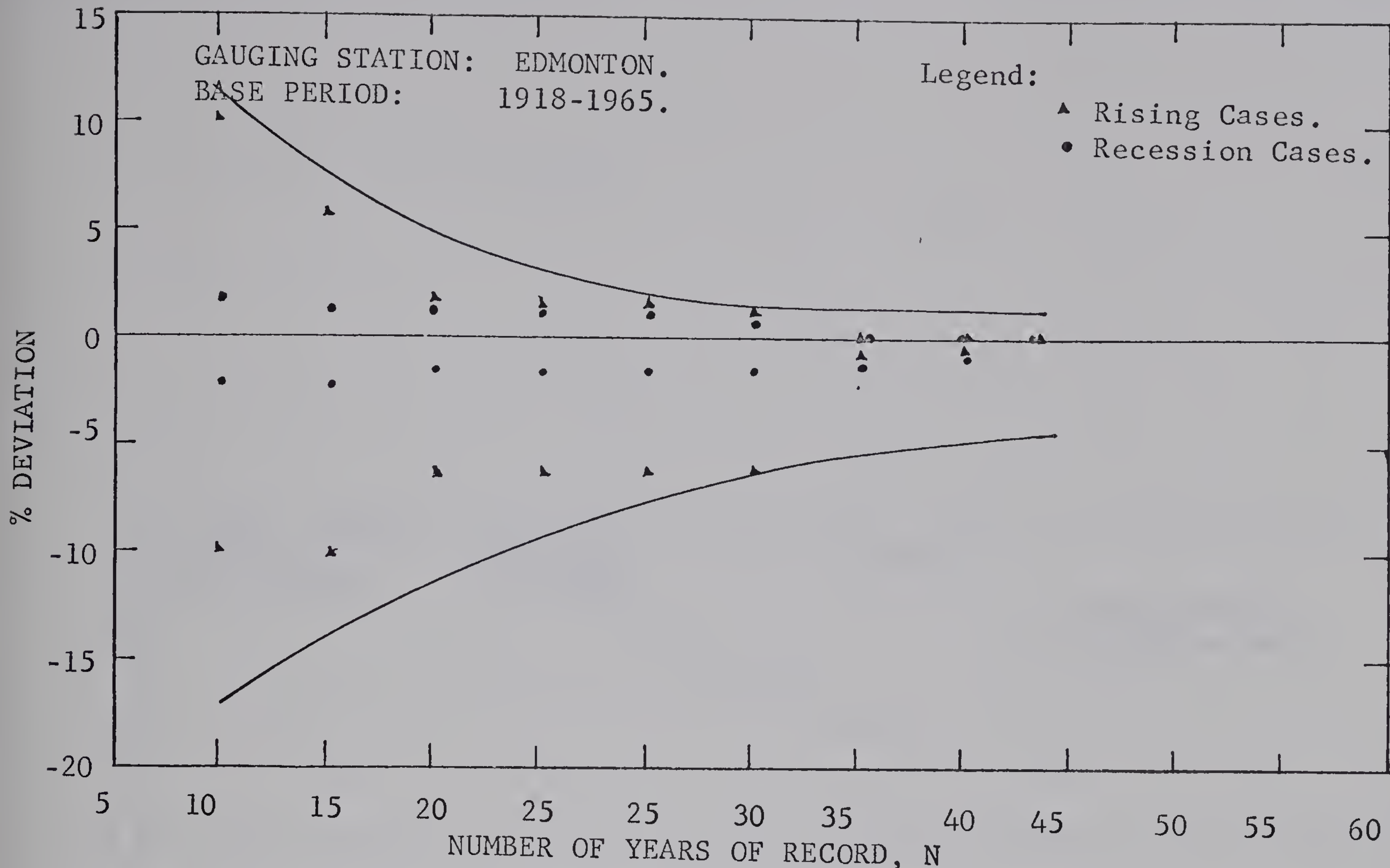


FIG. 5.11E MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=40000(CFS)

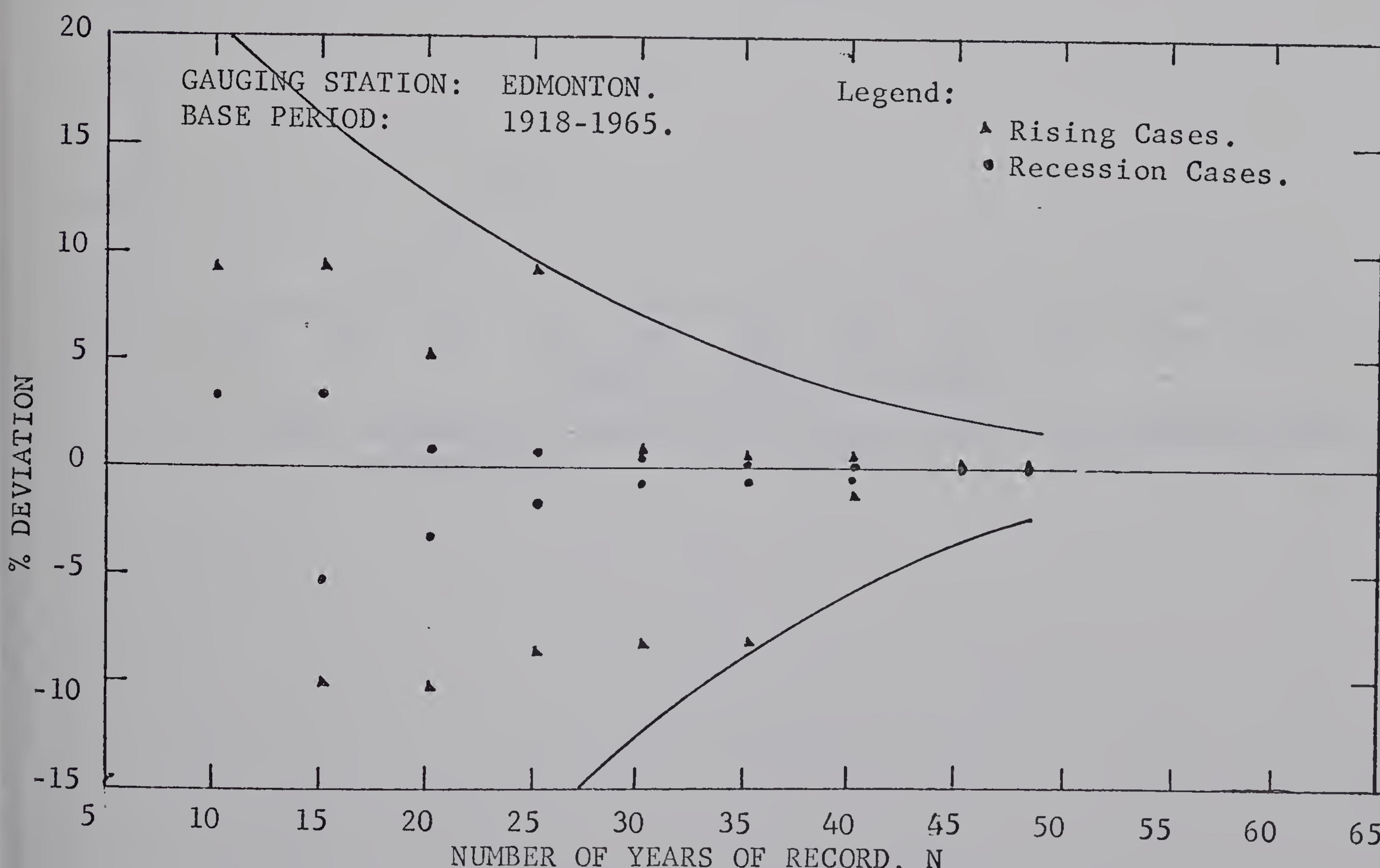


FIG. 5.11F MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=50000(CFS)

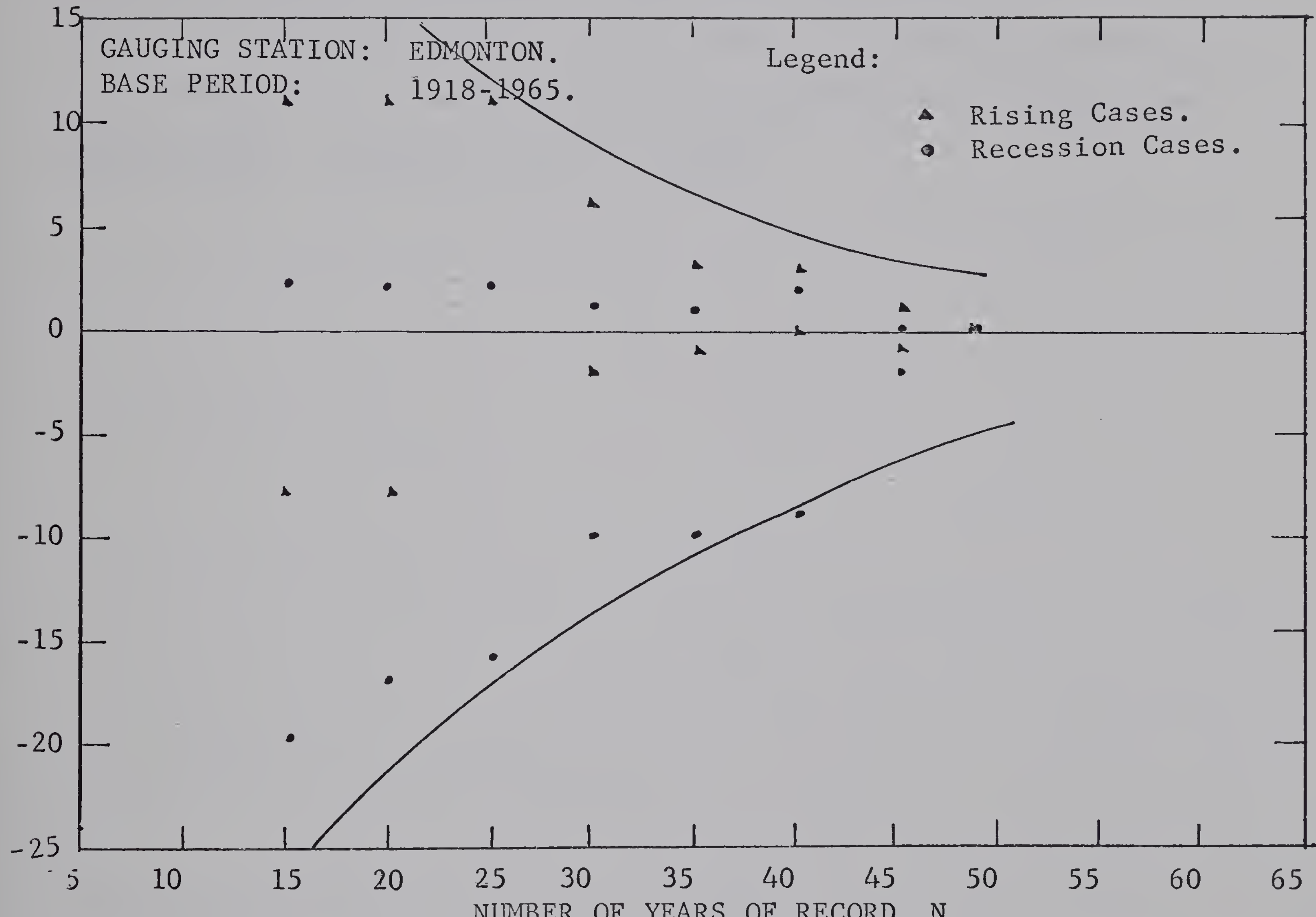


FIG. 5.11G MAXIMUM OF DEVIATIONS OF Q_{RATIO}-MEDIAN WITH RECORD LENGTH: D=60000(CFS)

5.7 Application of the Method

Where only stream flow records of comparatively shorter durations of about 15 years are available the need may arise to know the expected design flood hydrograph. In such cases, using the probability theory, a reliable estimate of the design flood can be made by the method explained in this thesis. The median recession curve obtained by the analysis is an alternate way of obtaining the characteristic recession curve of a basin.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. A method was suggested to arrive statistically at the shape of the flood hydrograph for the North Saskatchewan River at Rocky Mountain House, Edmonton, and Prince Albert.

2. The recession portion of the flood hydrograph becomes flatter for stations further downstream.

3. The combination of the median recession curve and the rising curve with a probability equal to the probability of occurrence of the flood peak simulate the actual hydrograph of the given probable flood quite well.

4. The median recession curve can be considered to be a characteristic curve for the given basin.

5. A record length of about 13 years was found to provide rising and recession curve of the hydrograph with sufficient accuracy to simulate the flood hydrograph.

6. The rising and recession curves allow computation of a design flood hydrograph when the volume of runoff, i.e. the excess precipitation is given, and the technique developed gives a better representation of the flood hydrograph than the unit hydrograph theory.

6.2 Recommendations

1. It is recommended that the analysis as described in this thesis be carried out for all gauging stations in a region with the objective of determining regional rising and recession curves that

can be used for ungauged streams to determine design flood hydrograph.

2. It is recommended that the analysis as described in this thesis be carried out considering floods during certain periods of the year only, with the objective of determining seasonal variations in the rising and recession curves.

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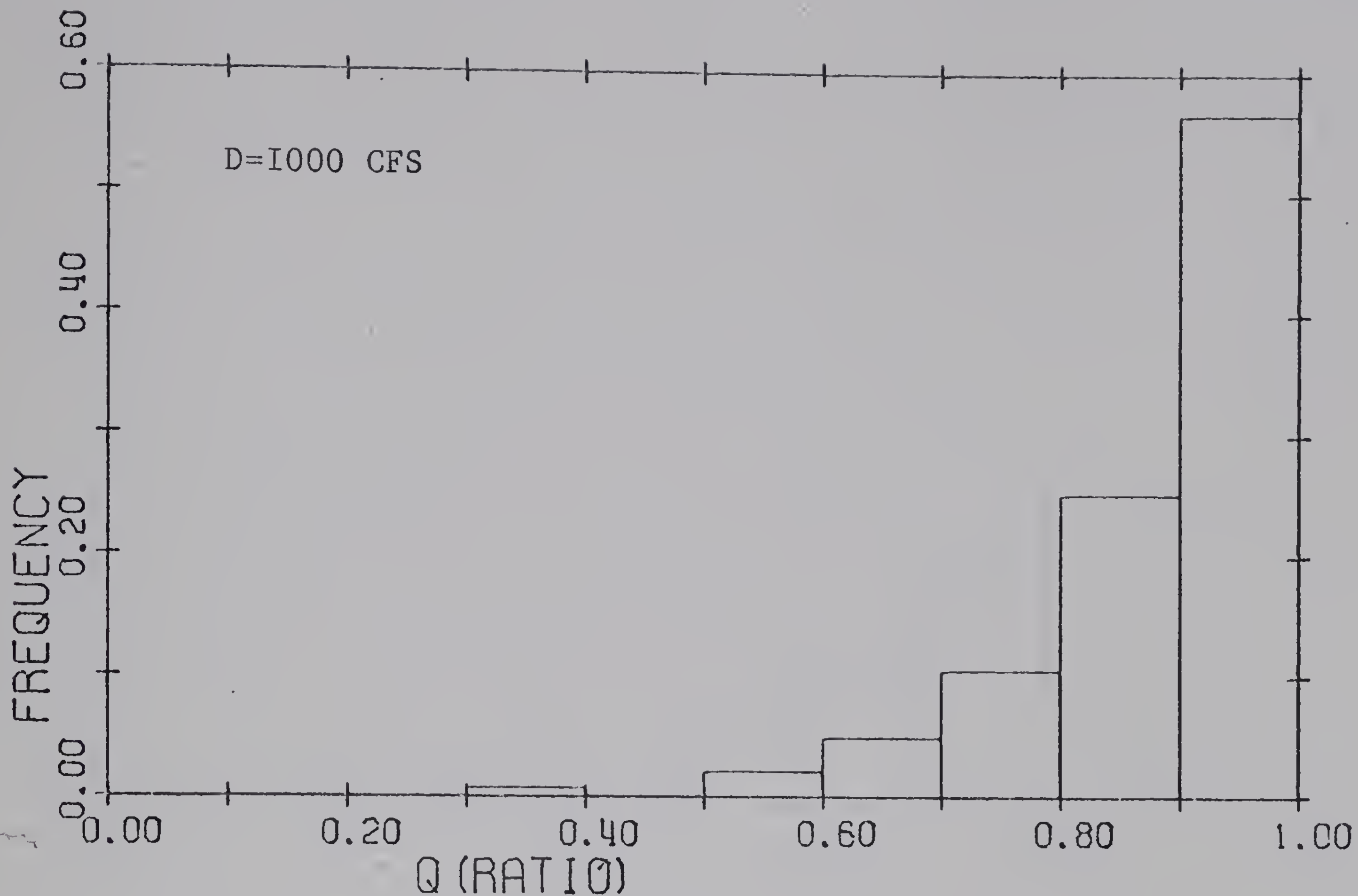
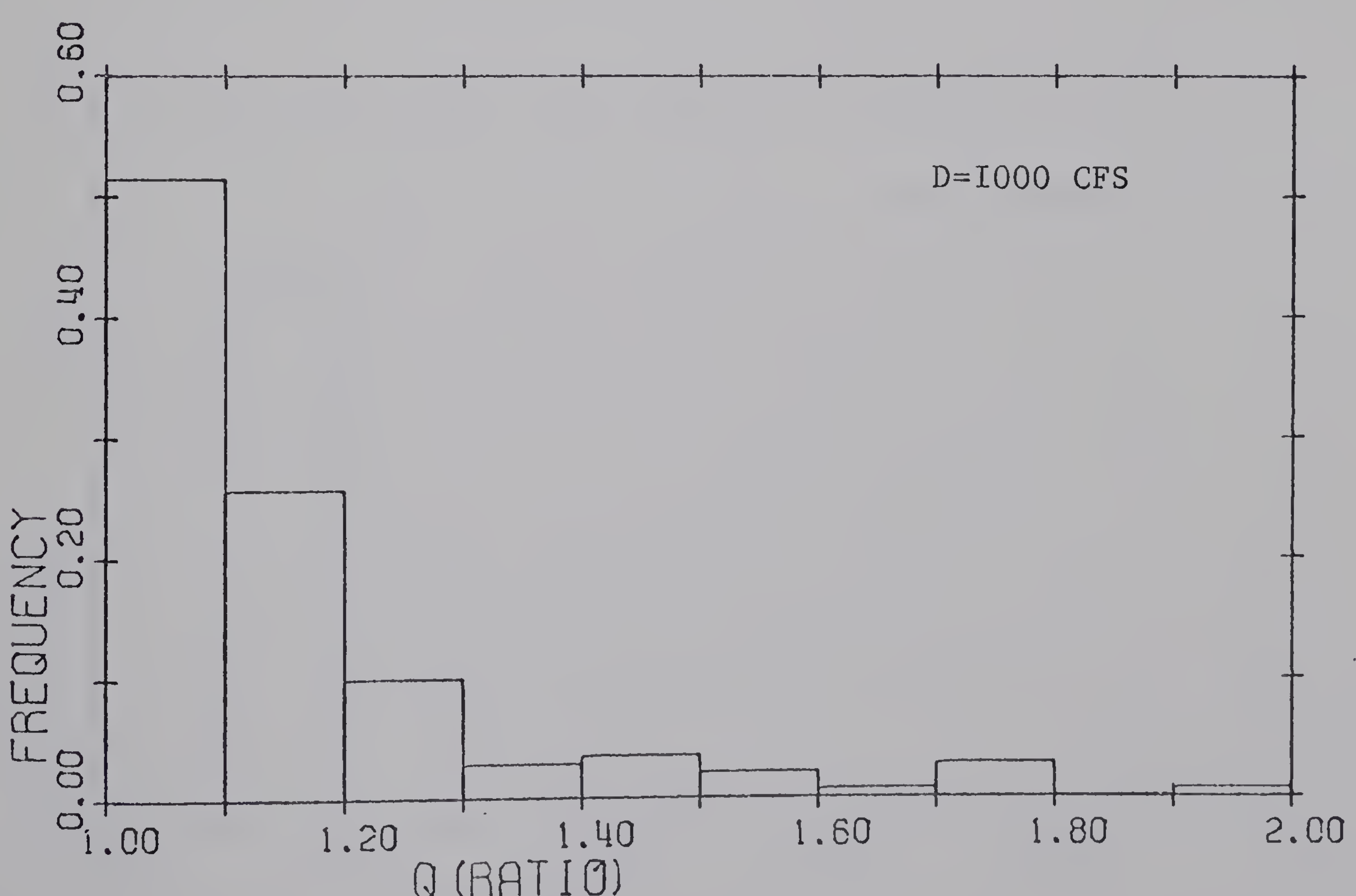
APPENDIX A

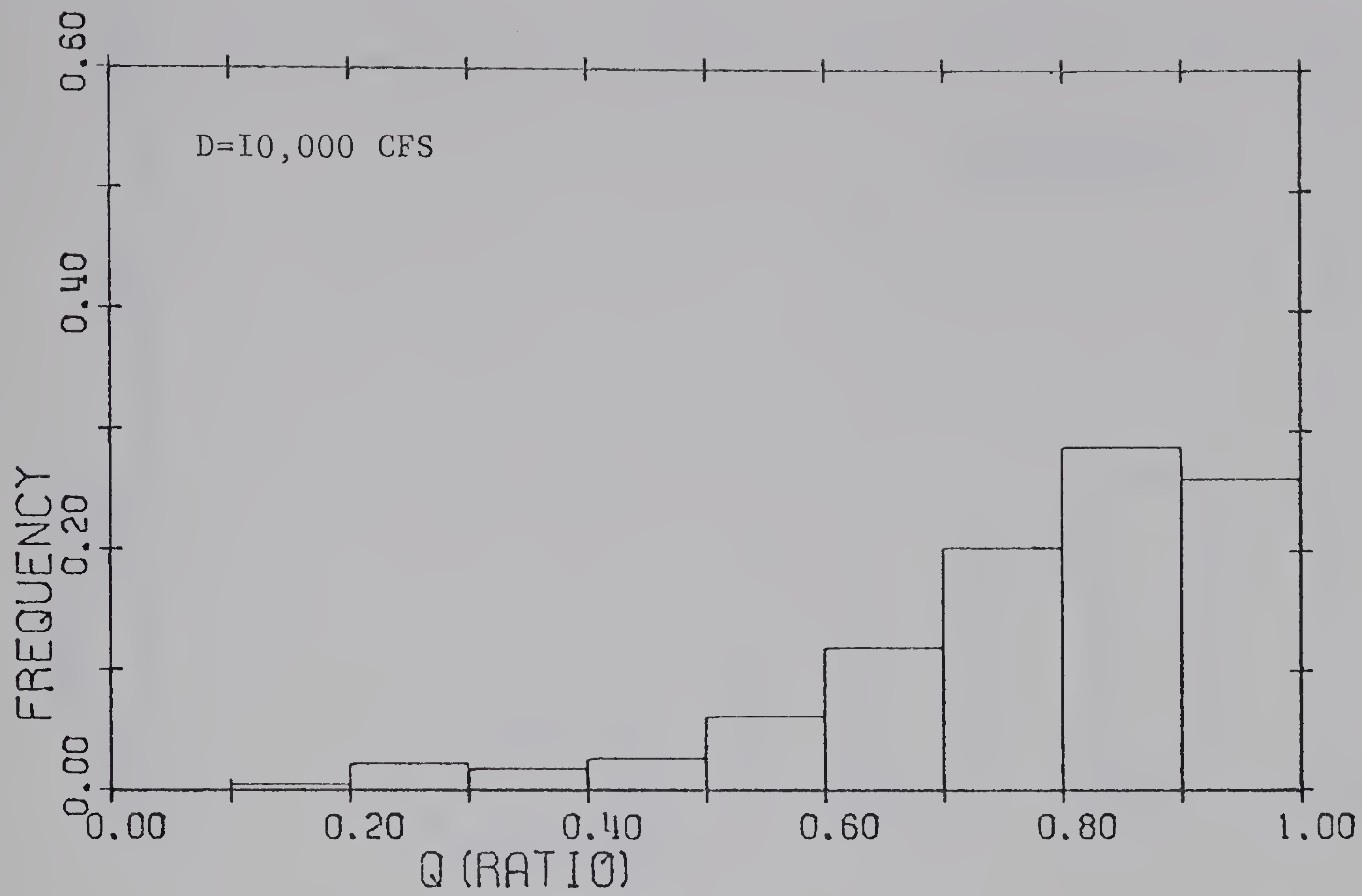
FREQUENCY HISTOGRAMS OF Q_{RATIO}.



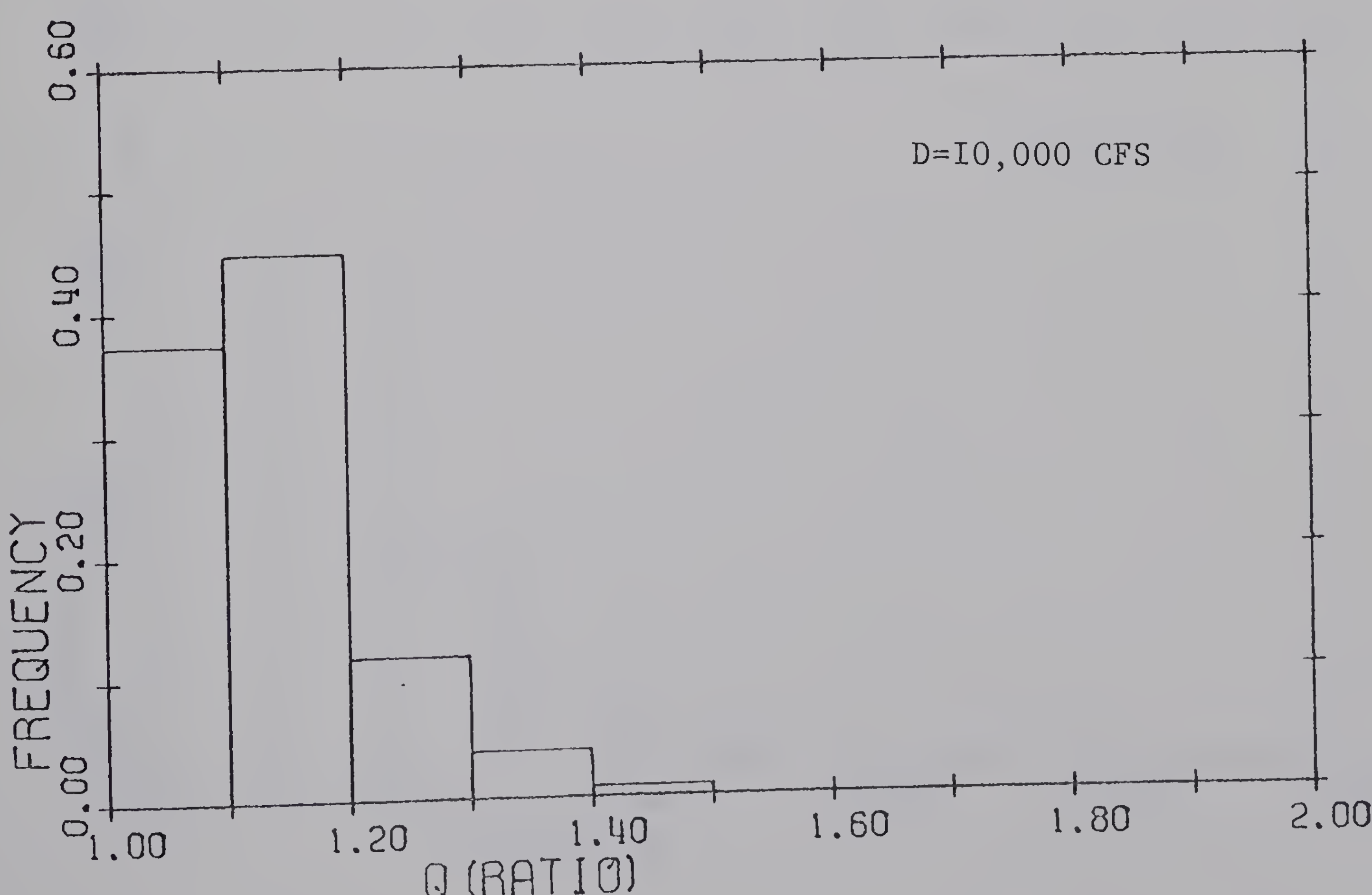
G. A-IA FREQUENCY HISTOGRAM OF Q (RATIO) : RISING CASES



G. A-IB FREQUENCY HISTOGRAM OF Q (RATIO) : RECESSION CASES



IG . A-2A FREQUENCY HISTOGRAM OF Q (RATIO) :RISING CASES



IG . A-2B FREQUENCY HISTOGRAM OF Q (RATIO) :RECESSION CASES

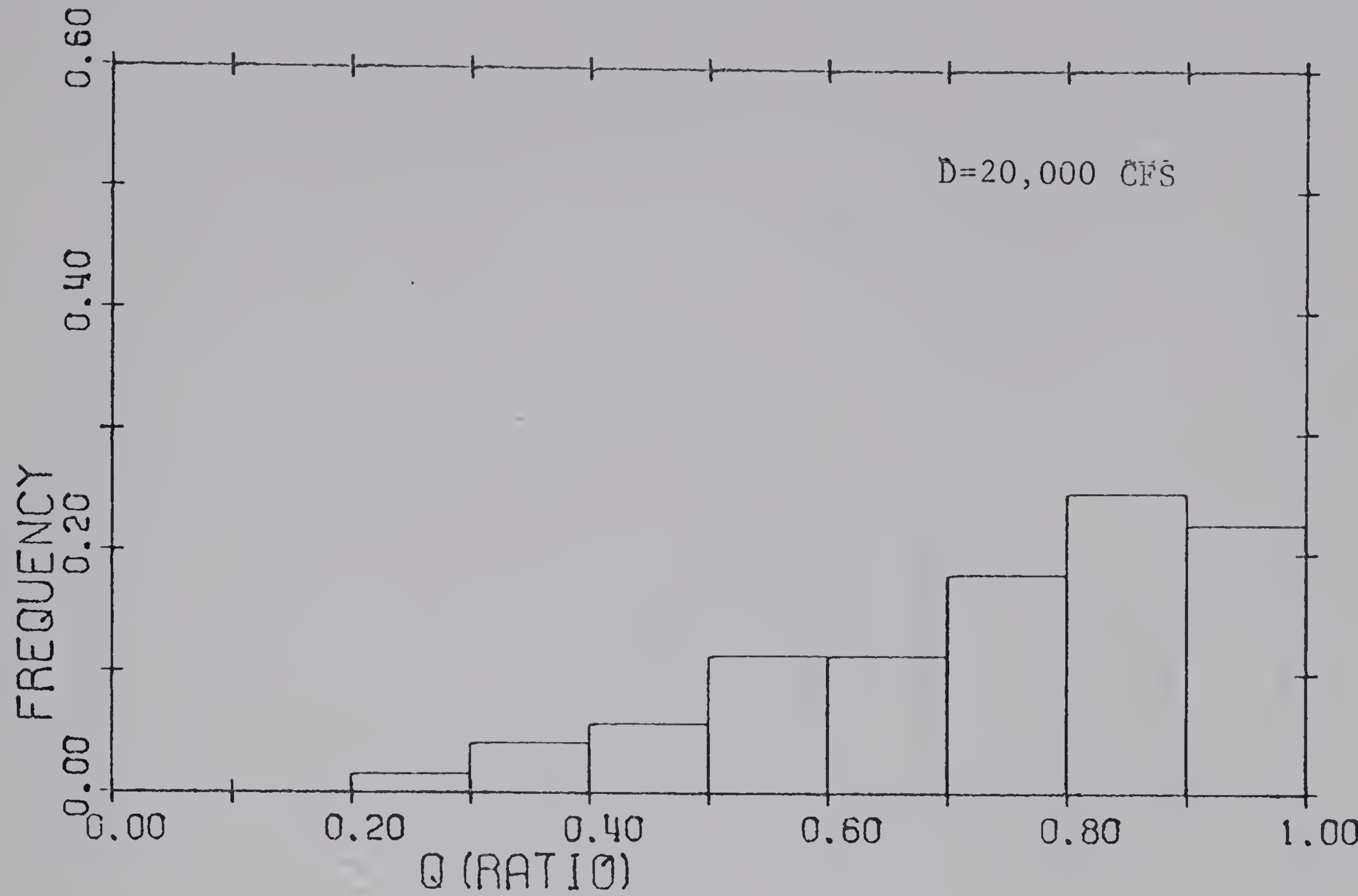
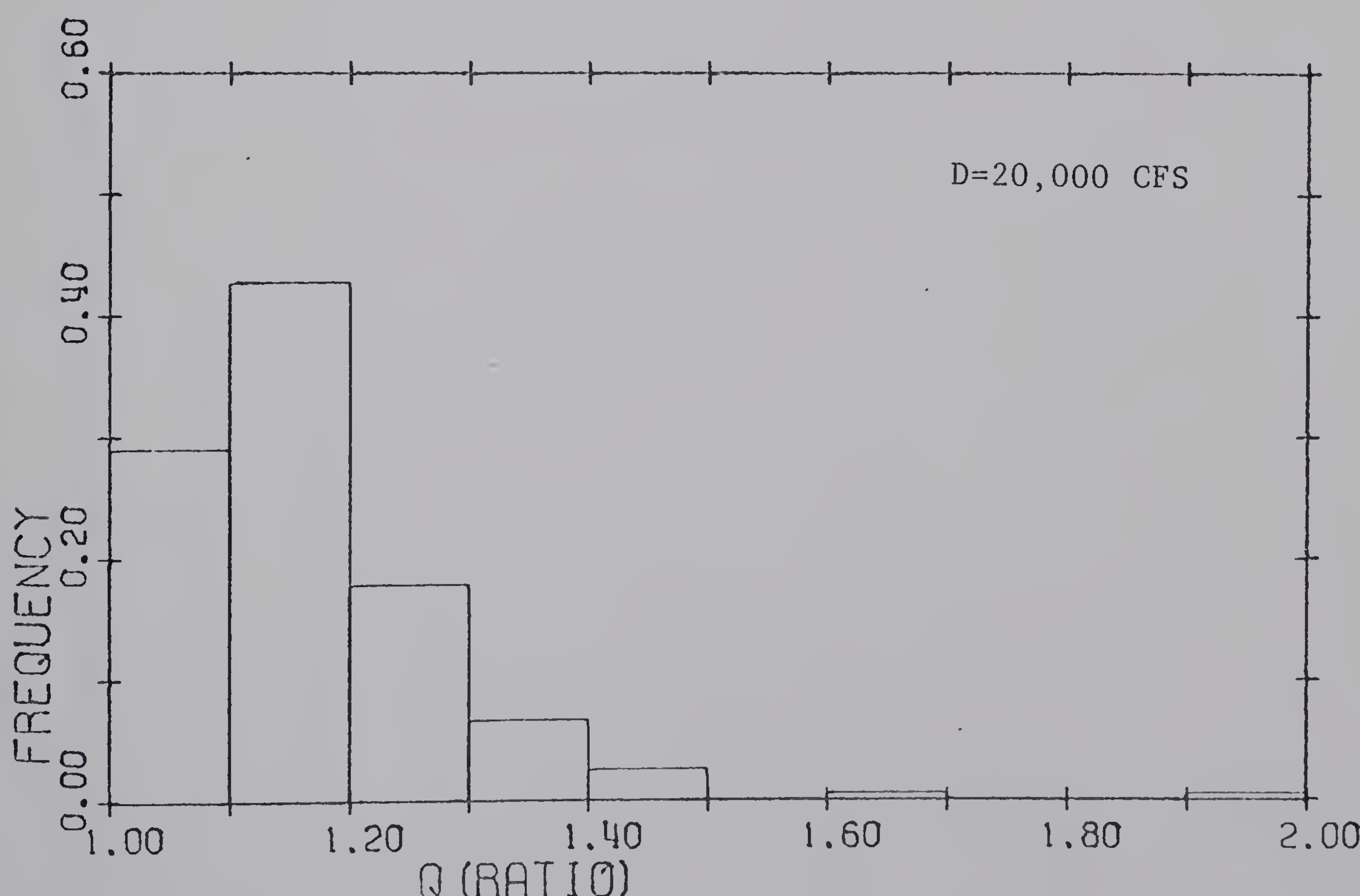
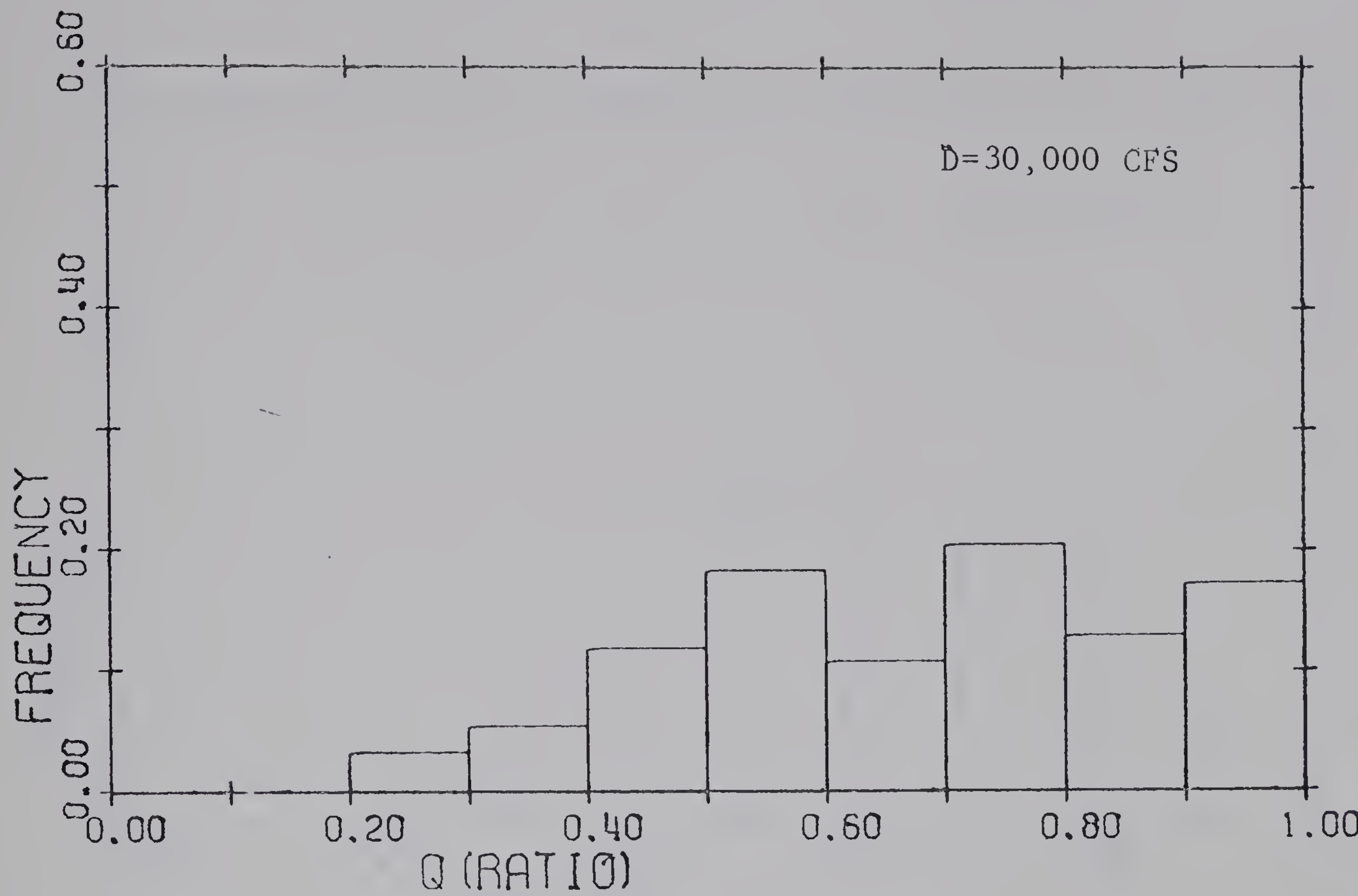


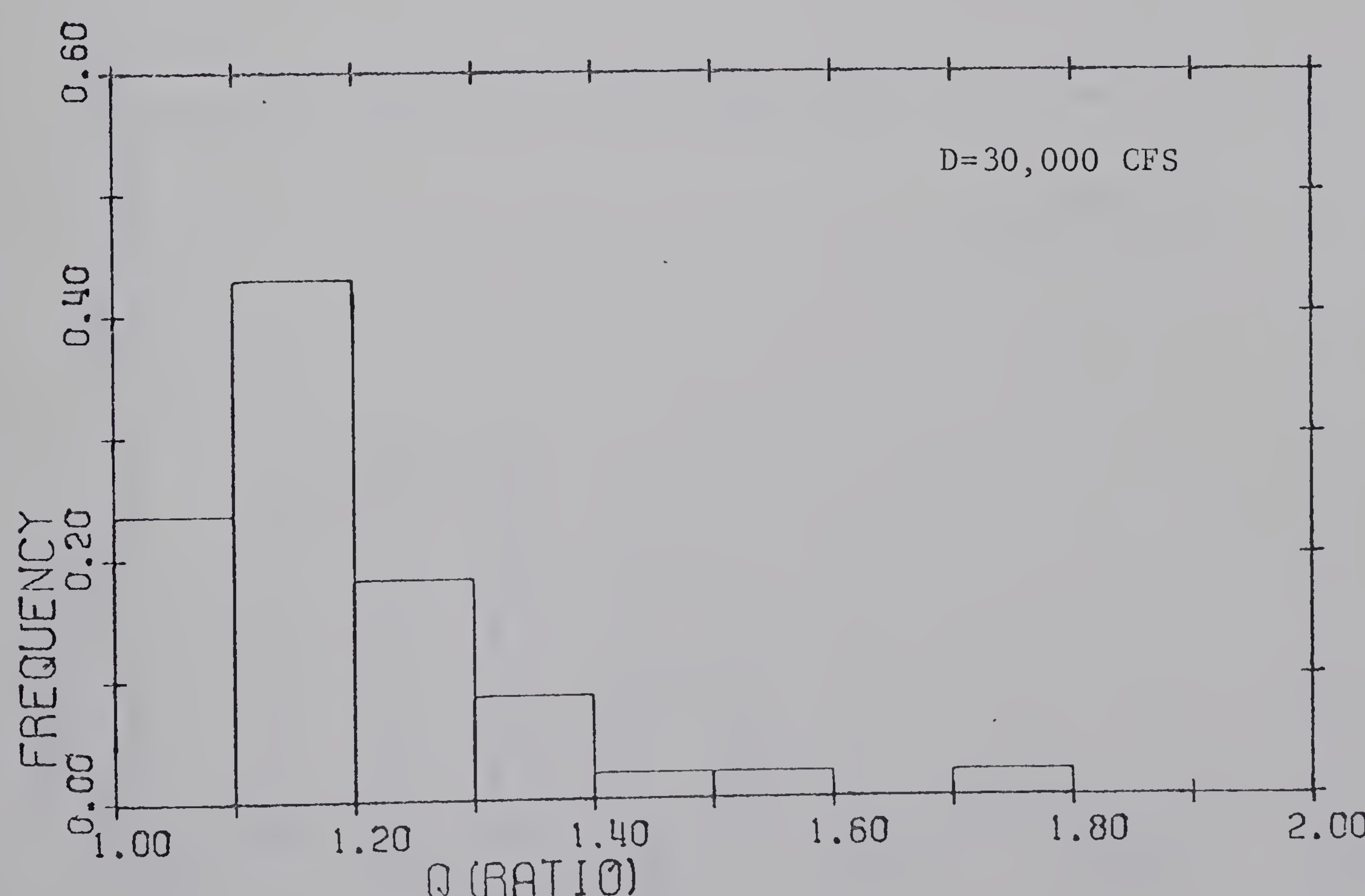
FIG. A-3A FREQUENCY HISTOGRAM OF Q (RATIO) : RISING CASES



. A3B FREQUENCY HISTOGRAM OF Q (RATIO) : RECESSION CASES



IG. A-4A FREQUENCY HISTOGRAM OF Q (RATIO) :RISING CASES



G. A-4B FREQUENCY HISTOGRAM OF Q (RATIO) :RECESSION CASES

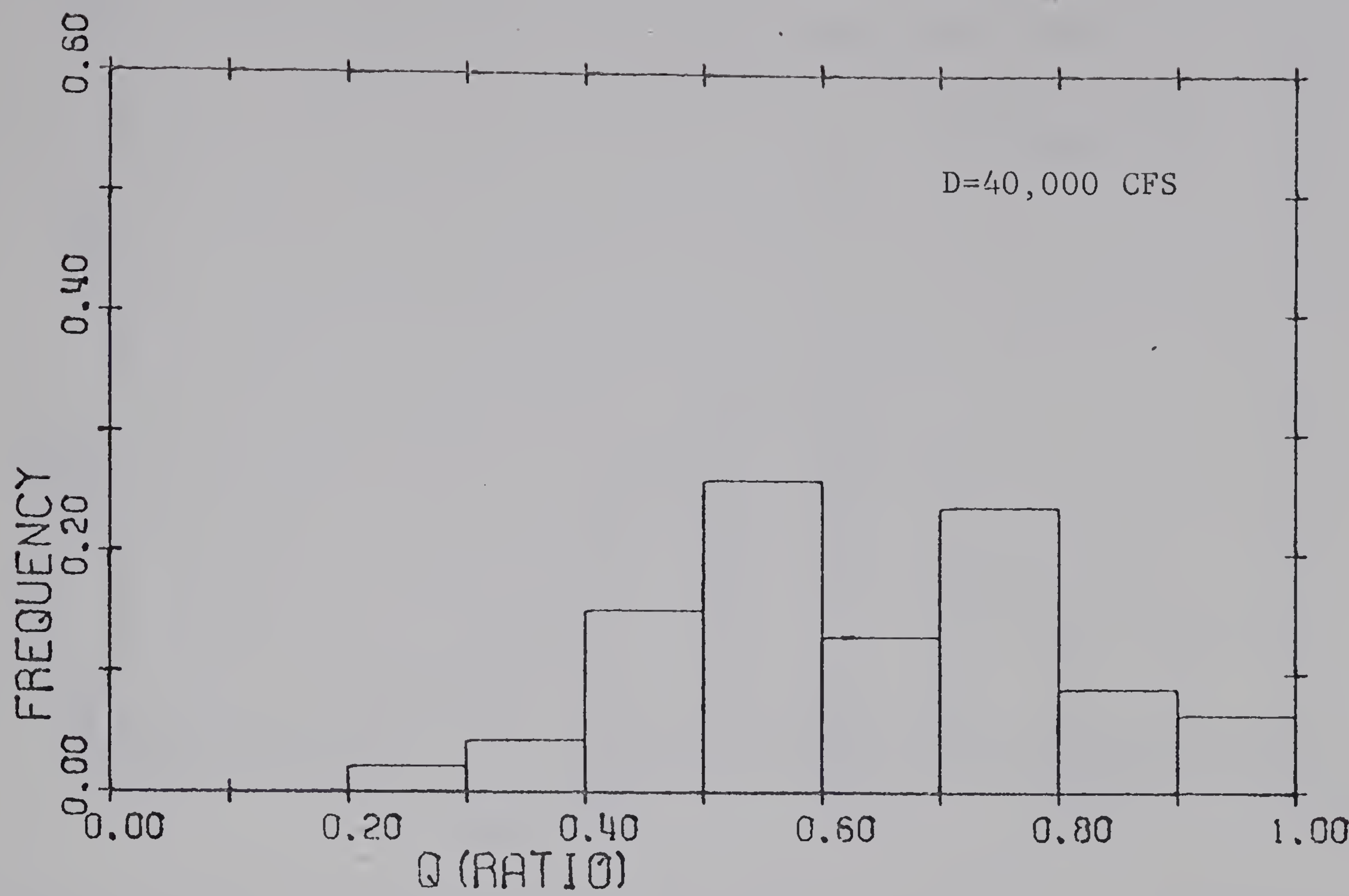
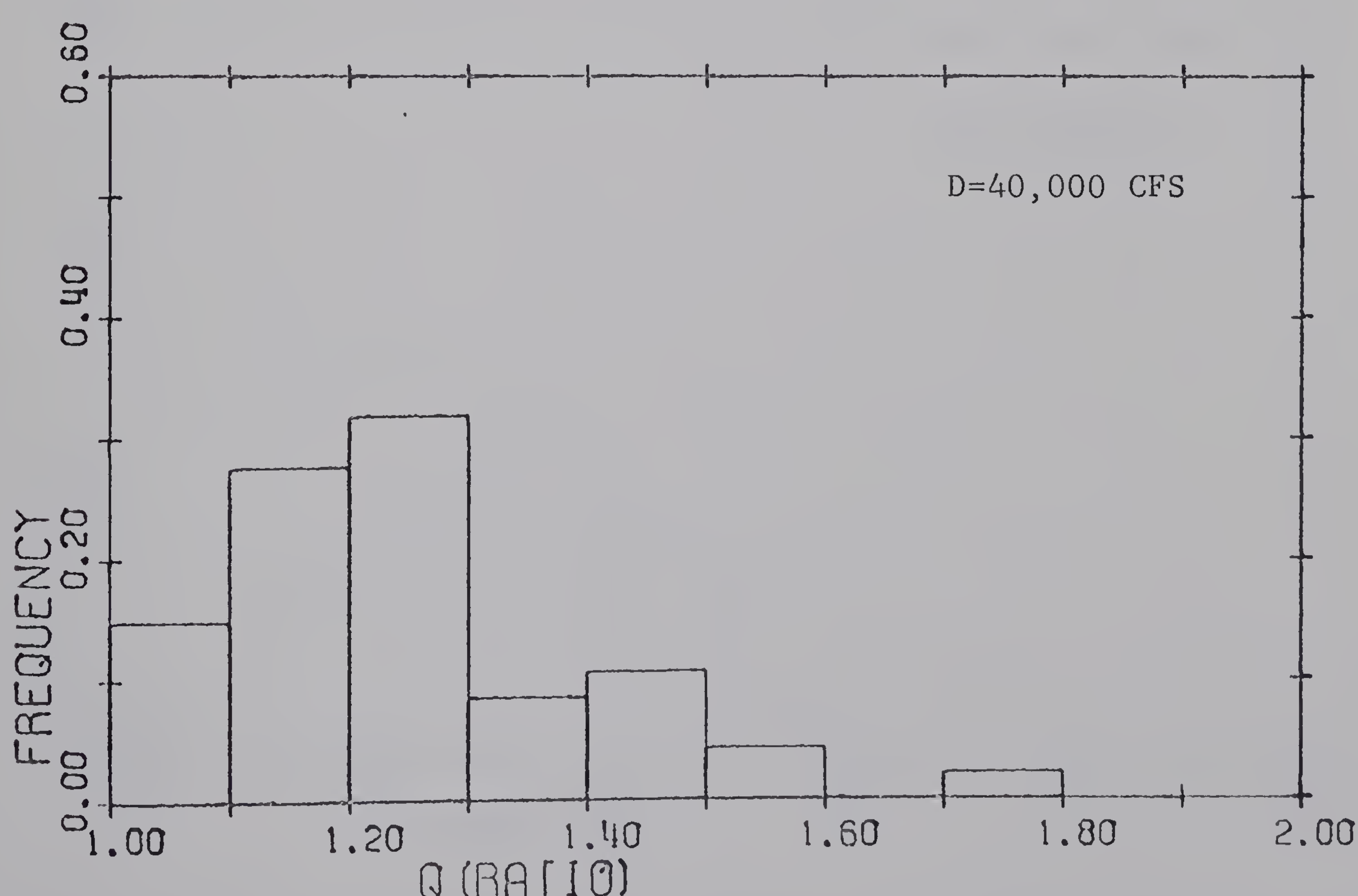


FIG. A-5A FREQUENCY HISTOGRAM OF Q (RATIO) : RISING CASES



G. A-5B FREQUENCY HISTOGRAM OF Q (RATIO) : RECESSION CASES

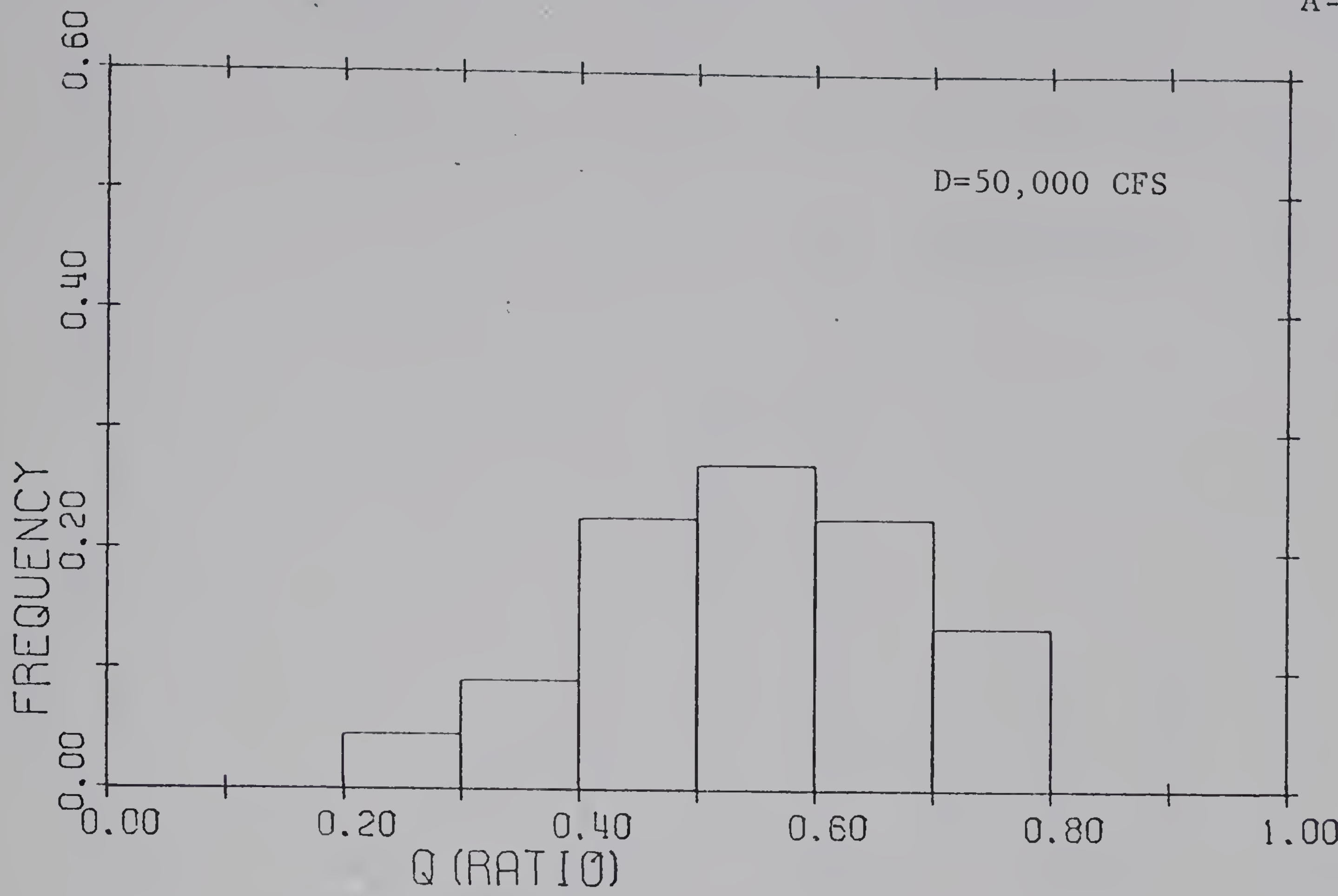


FIG. A-6A FREQUENCY HISTOGRAM OF Q (RATIO) : RISING CASES

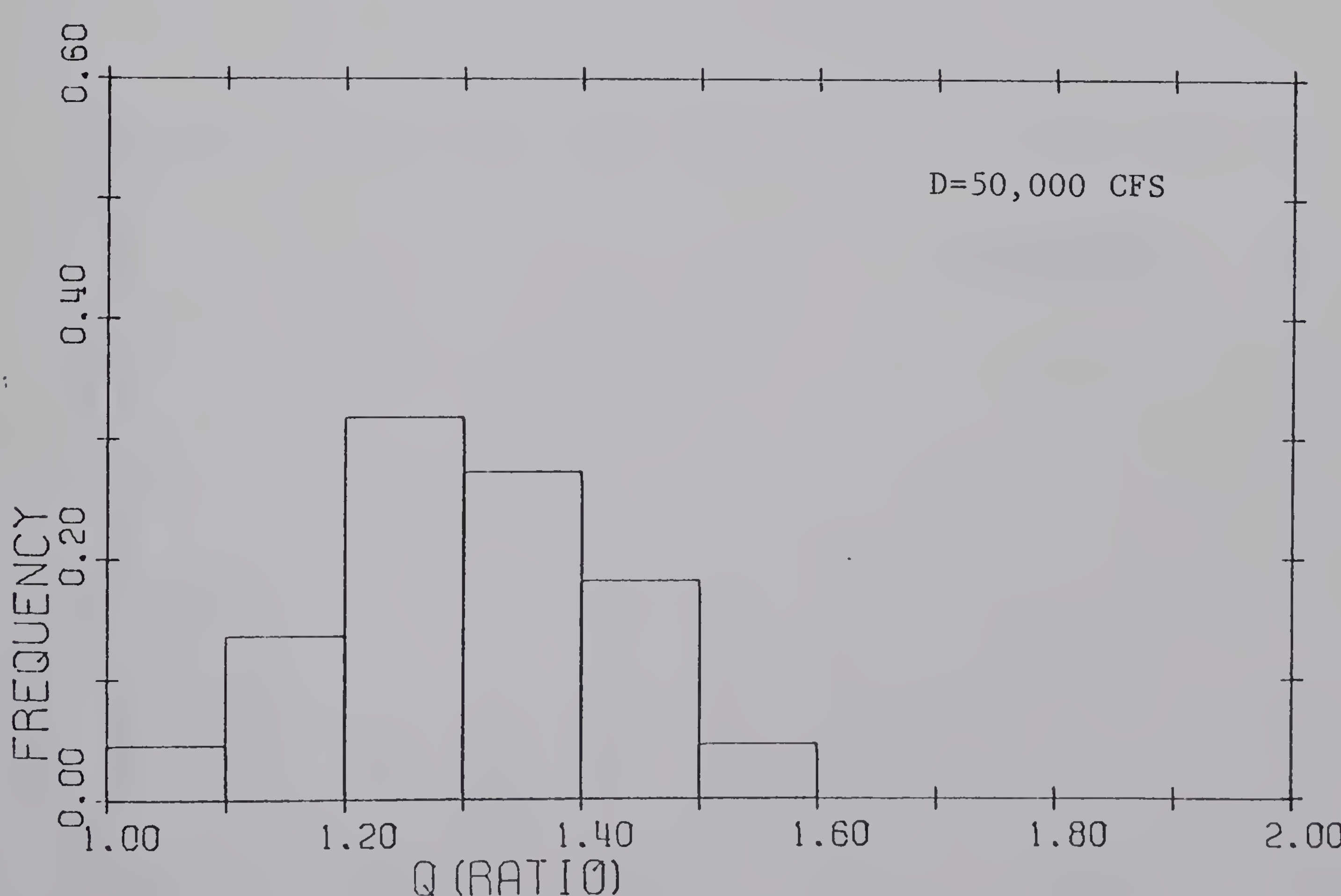


FIG. A-6B FREQUENCY HISTOGRAM OF Q (RATIO) : RECESSION CASES

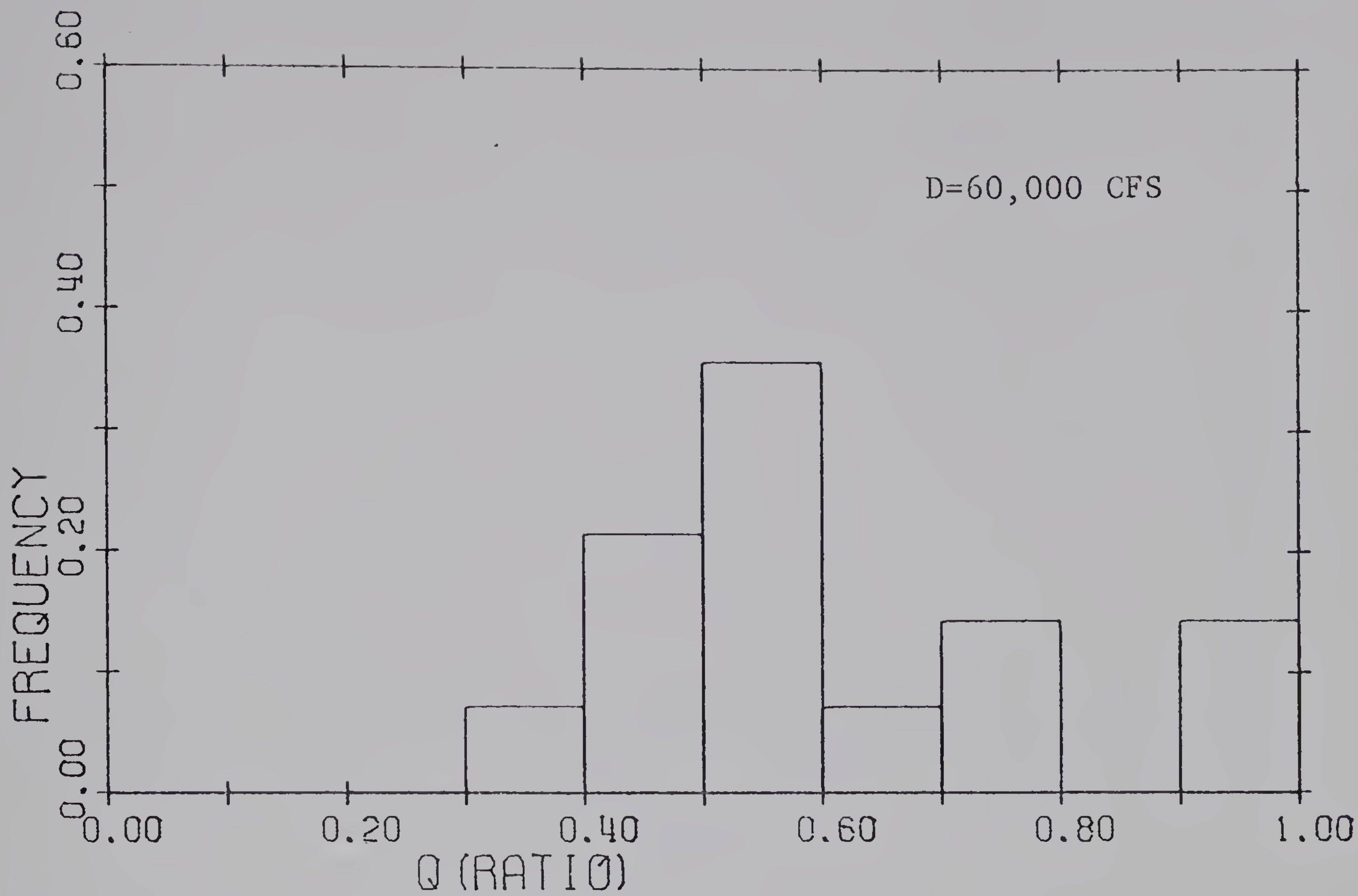


FIG. A-7A FREQUENCY HISTOGRAM OF Q (RATIO) : RISING CASES

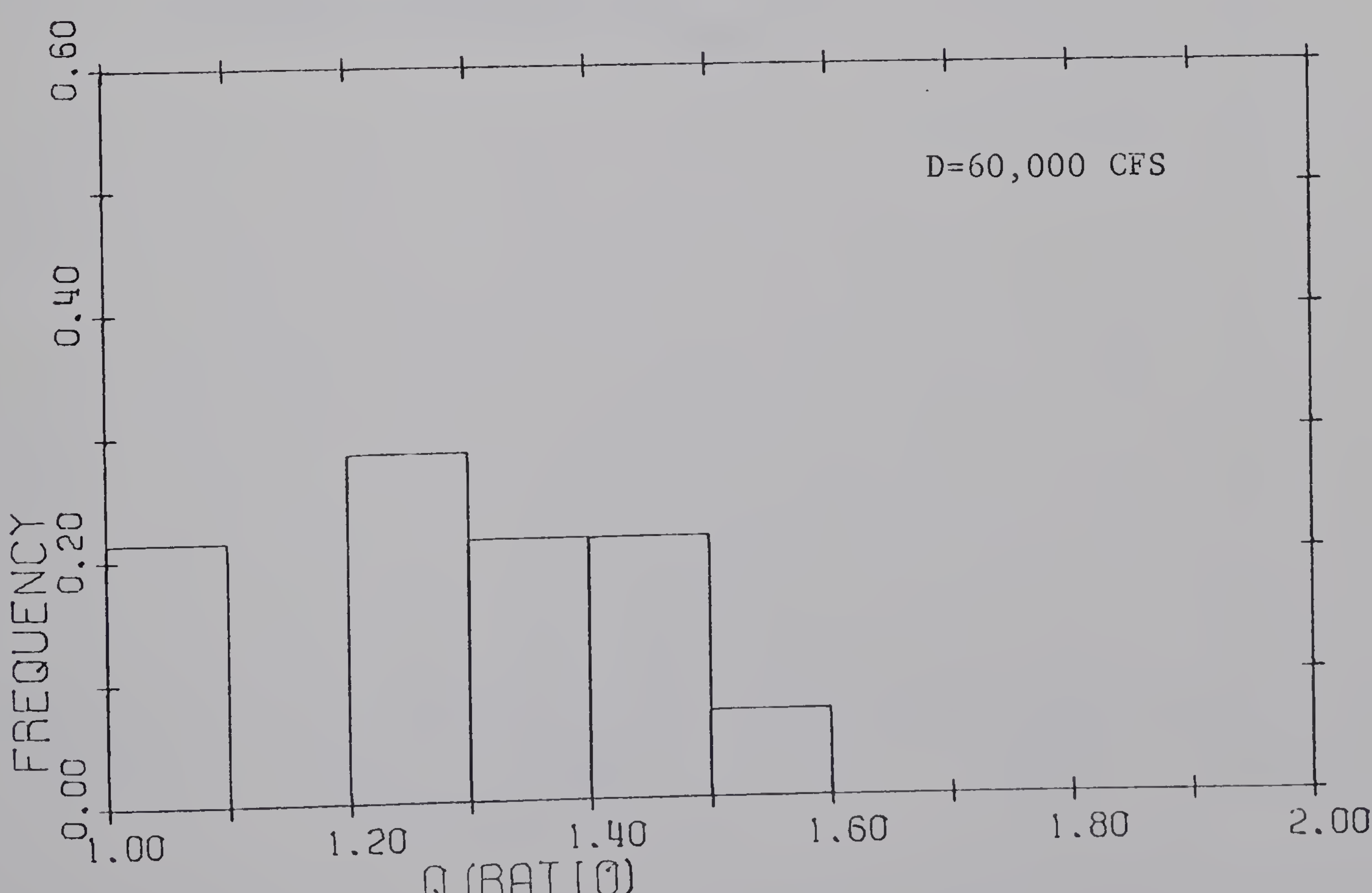
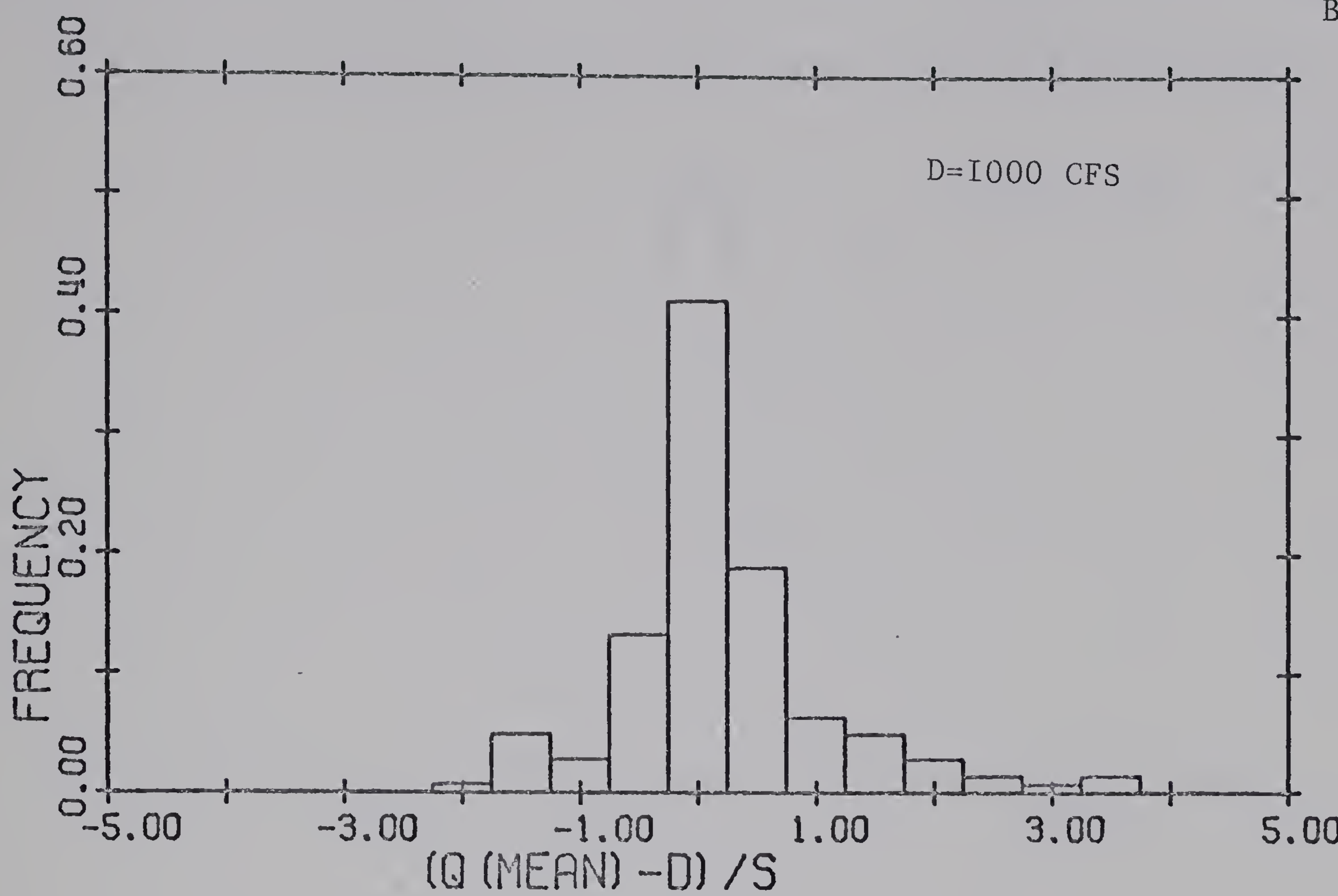


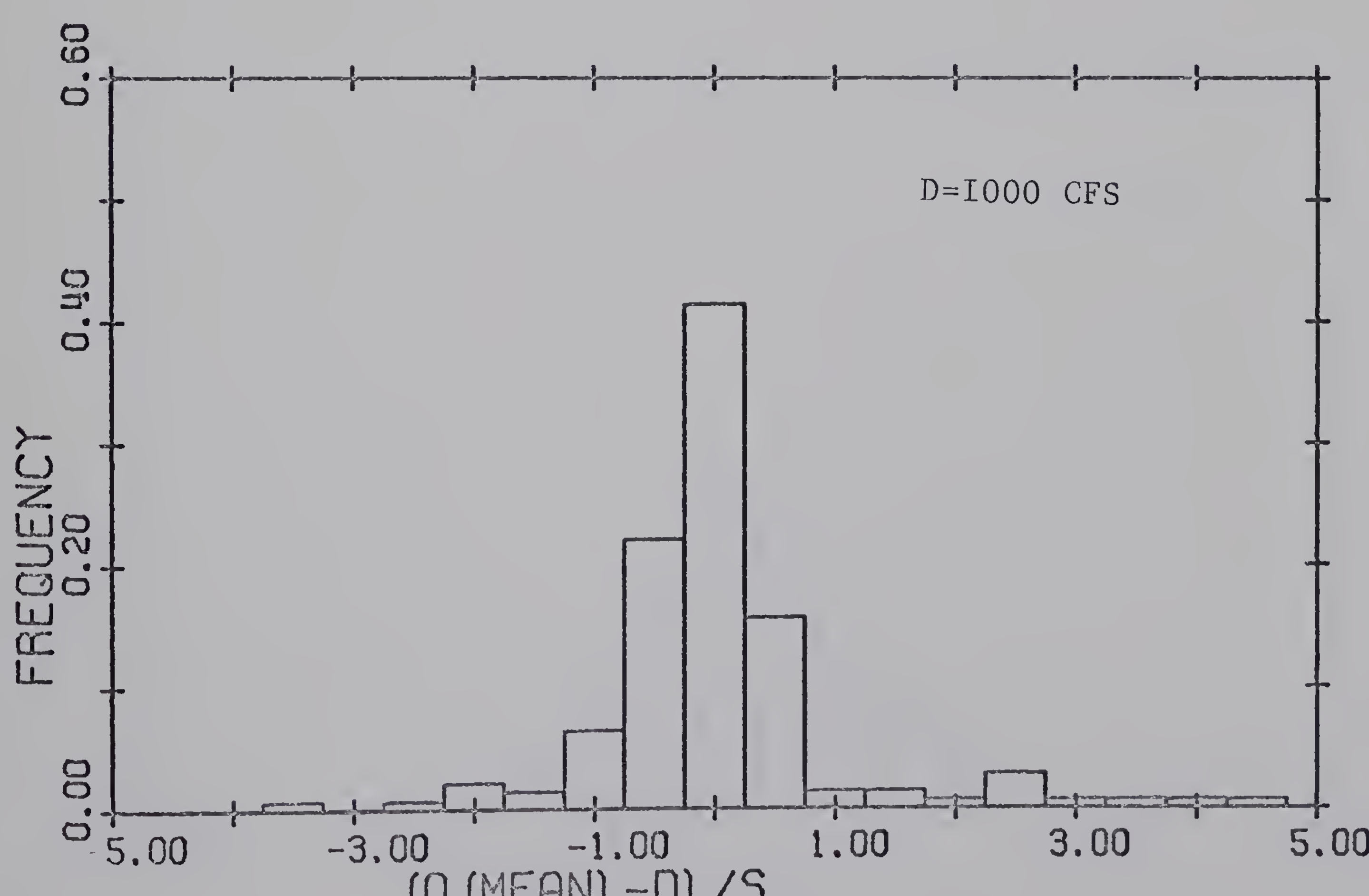
FIG. A-7B FREQUENCY HISTOGRAM OF Q (RATIO) : RECESSION CASES

APPENDIX B

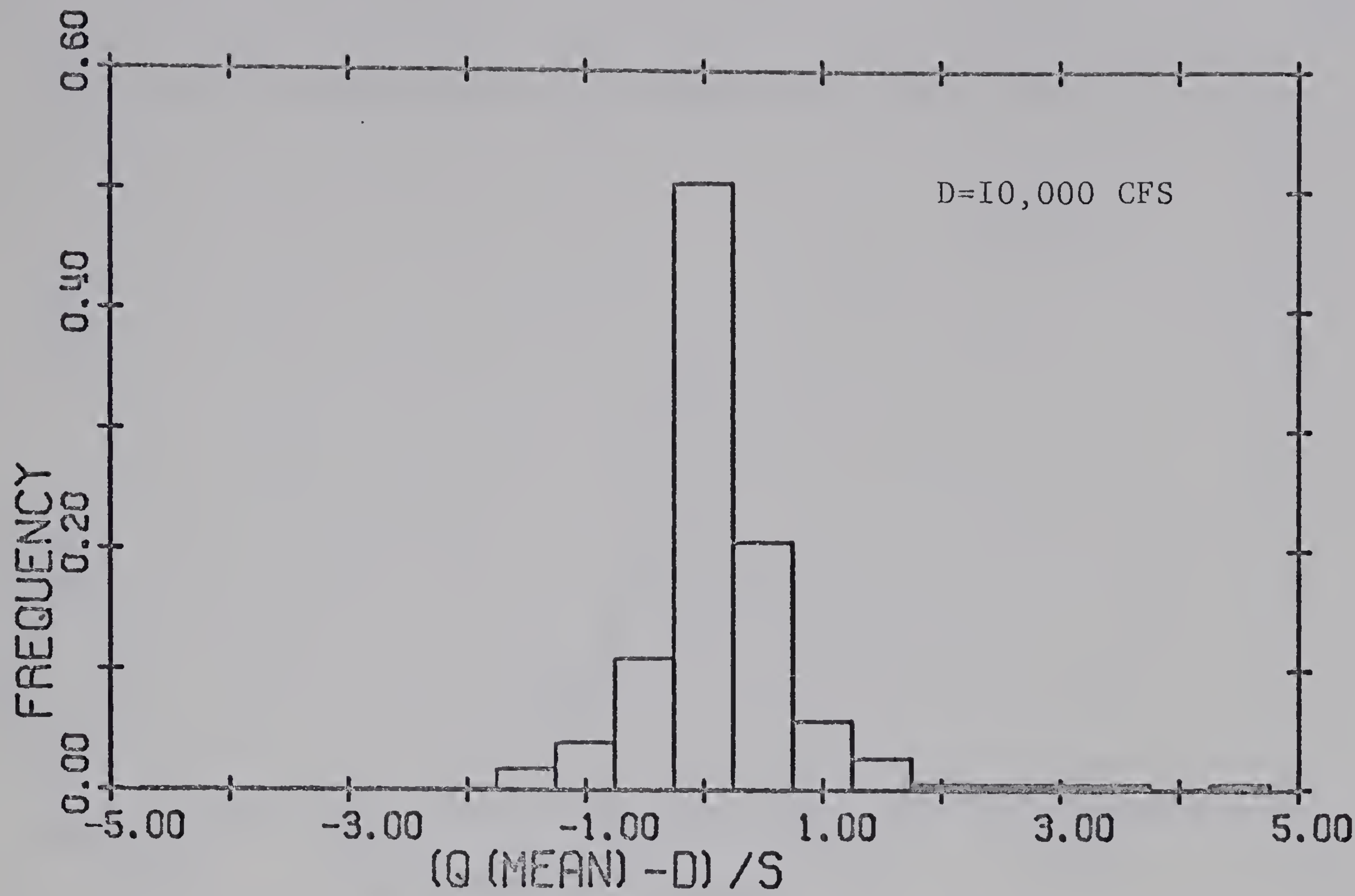
FREQUENCY HISTOGRAMS OF Q_{MEAN}.



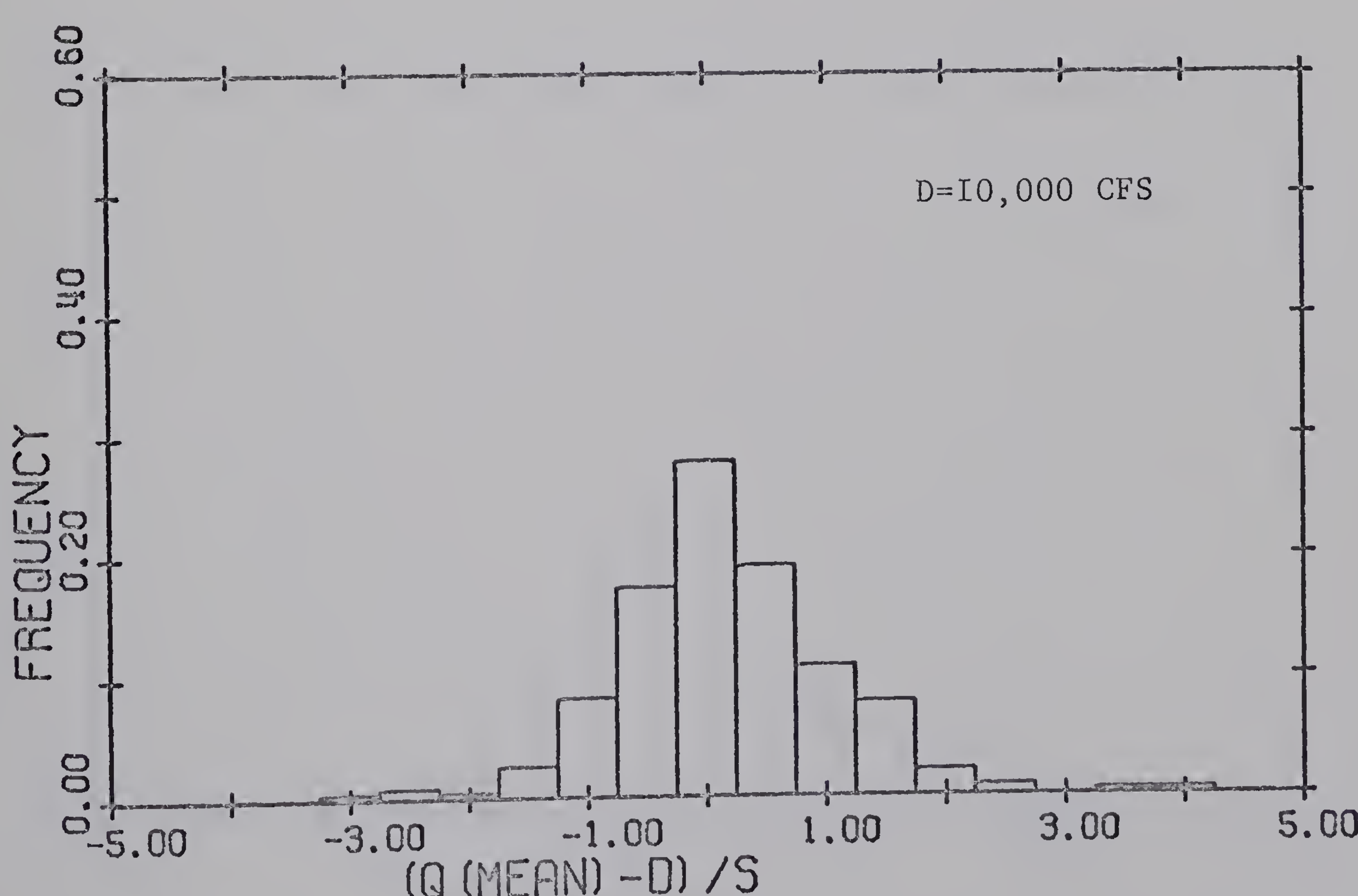
IG. B-IA FREQUENCY HISTOGRAM OF Q (MEAN) :RISING CASES



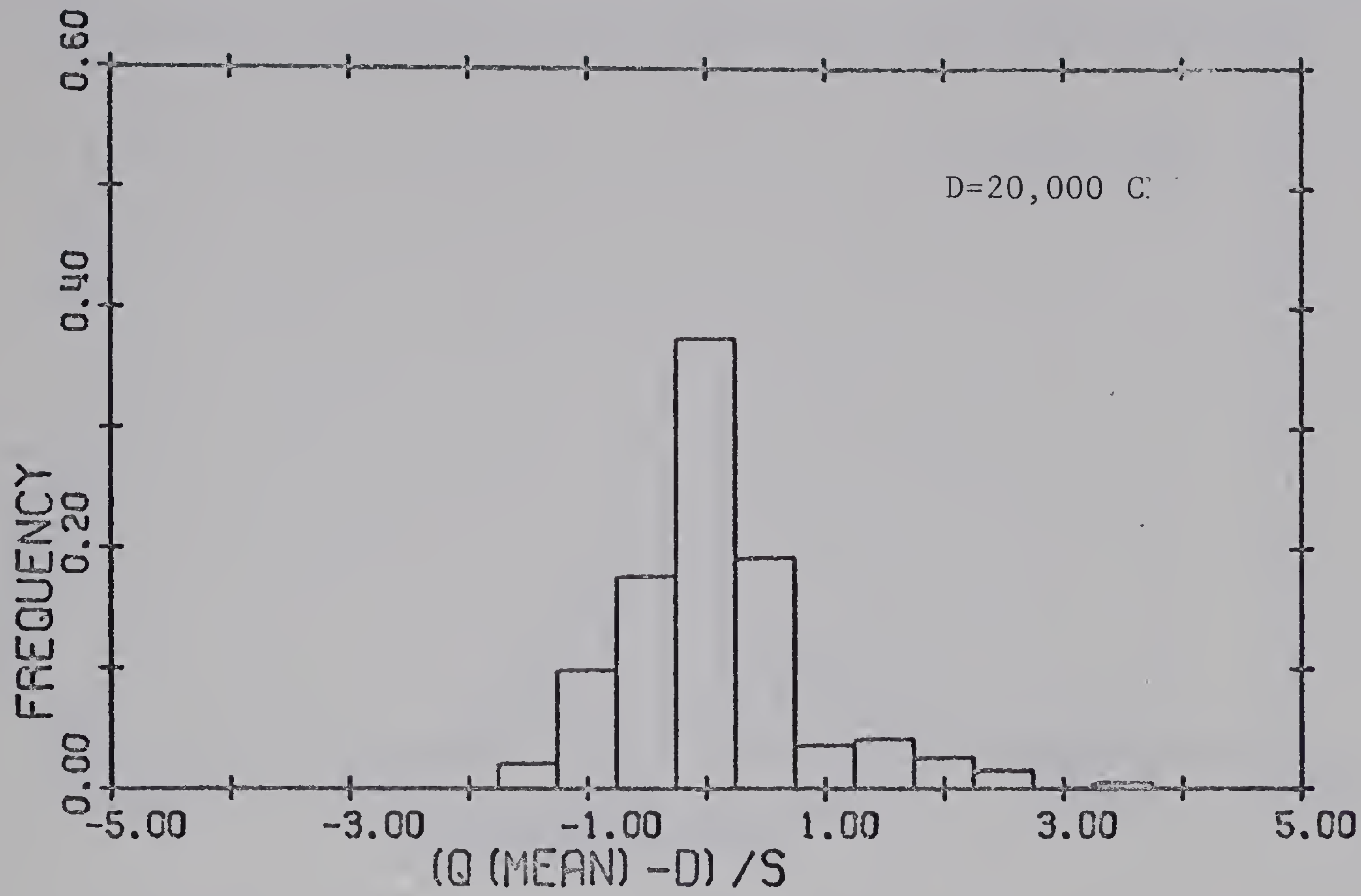
IG. B-IB FREQUENCY HISTOGRAM OF Q (MEAN) :RECESSION CASES



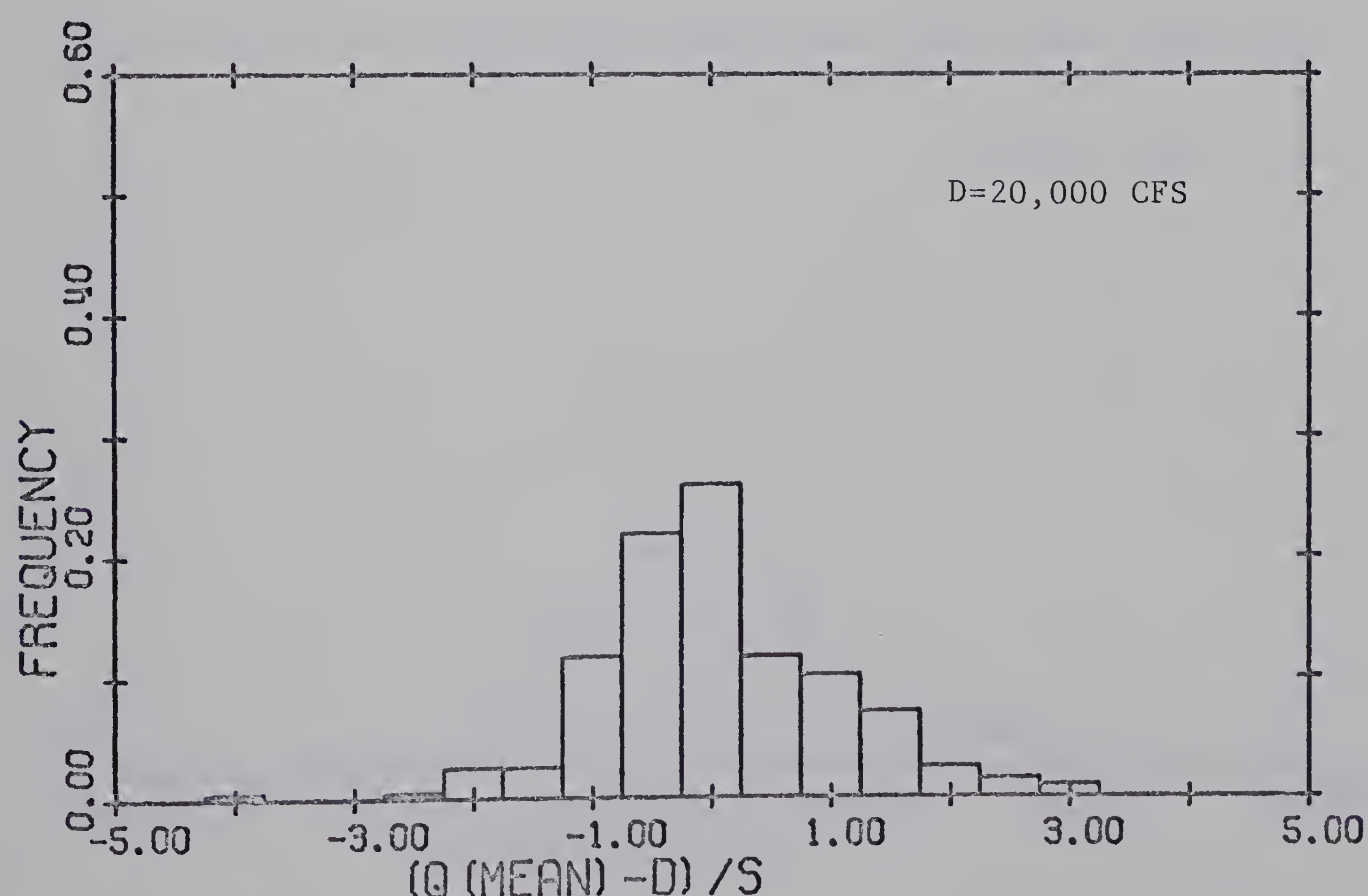
G. B-2A FREQUENCY HISTOGRAM OF Q (MEAN) : RISING CASES



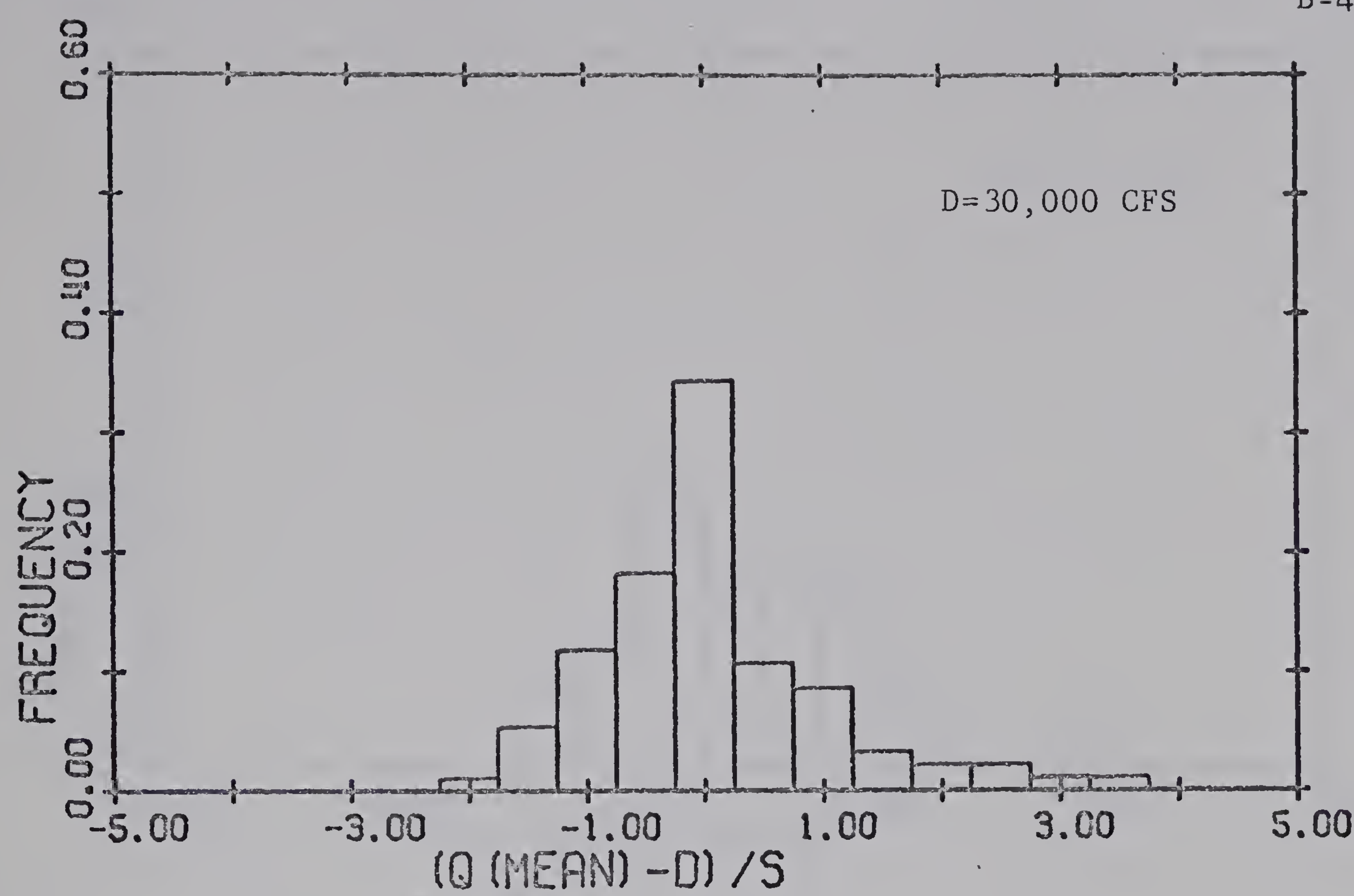
G. B-2B FREQUENCY HISTOGRAM OF Q (MEAN) : RECESSION CASES



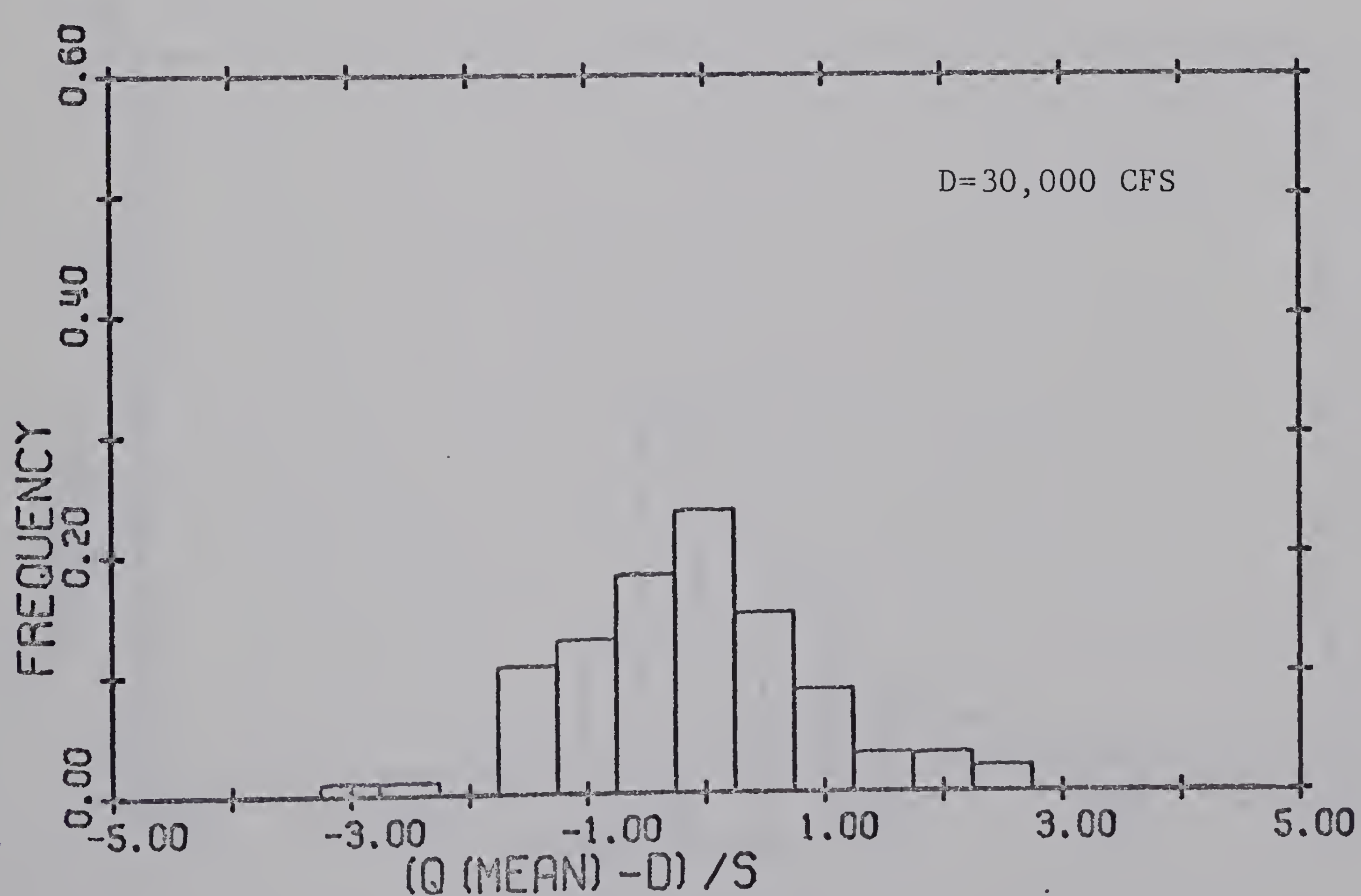
B-3A FREQUENCY HISTOGRAM OF Q (MEAN) : RISING CASES



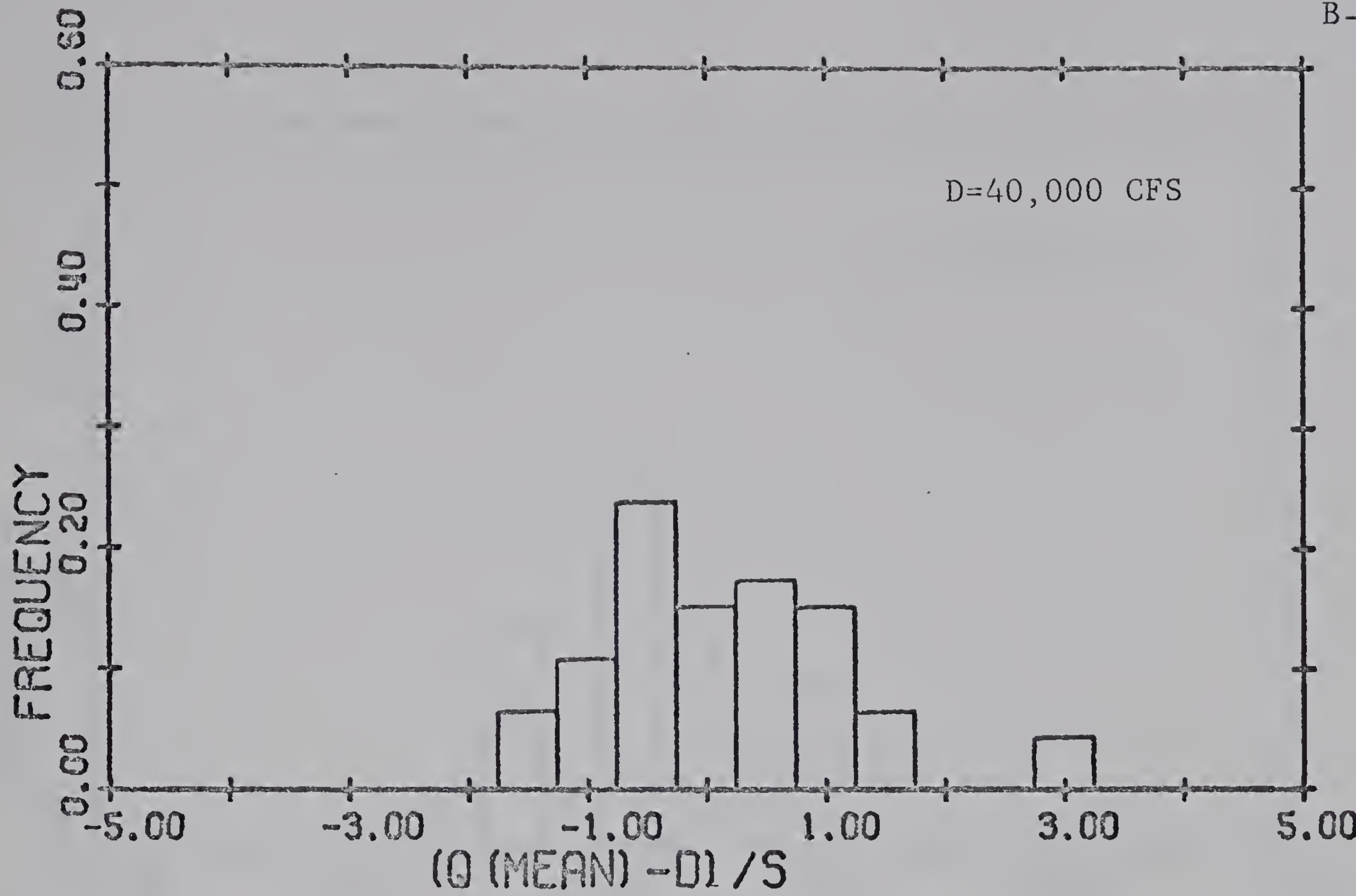
B-3B FREQUENCY HISTOGRAM OF Q (MEAN) : RECESSION CASES



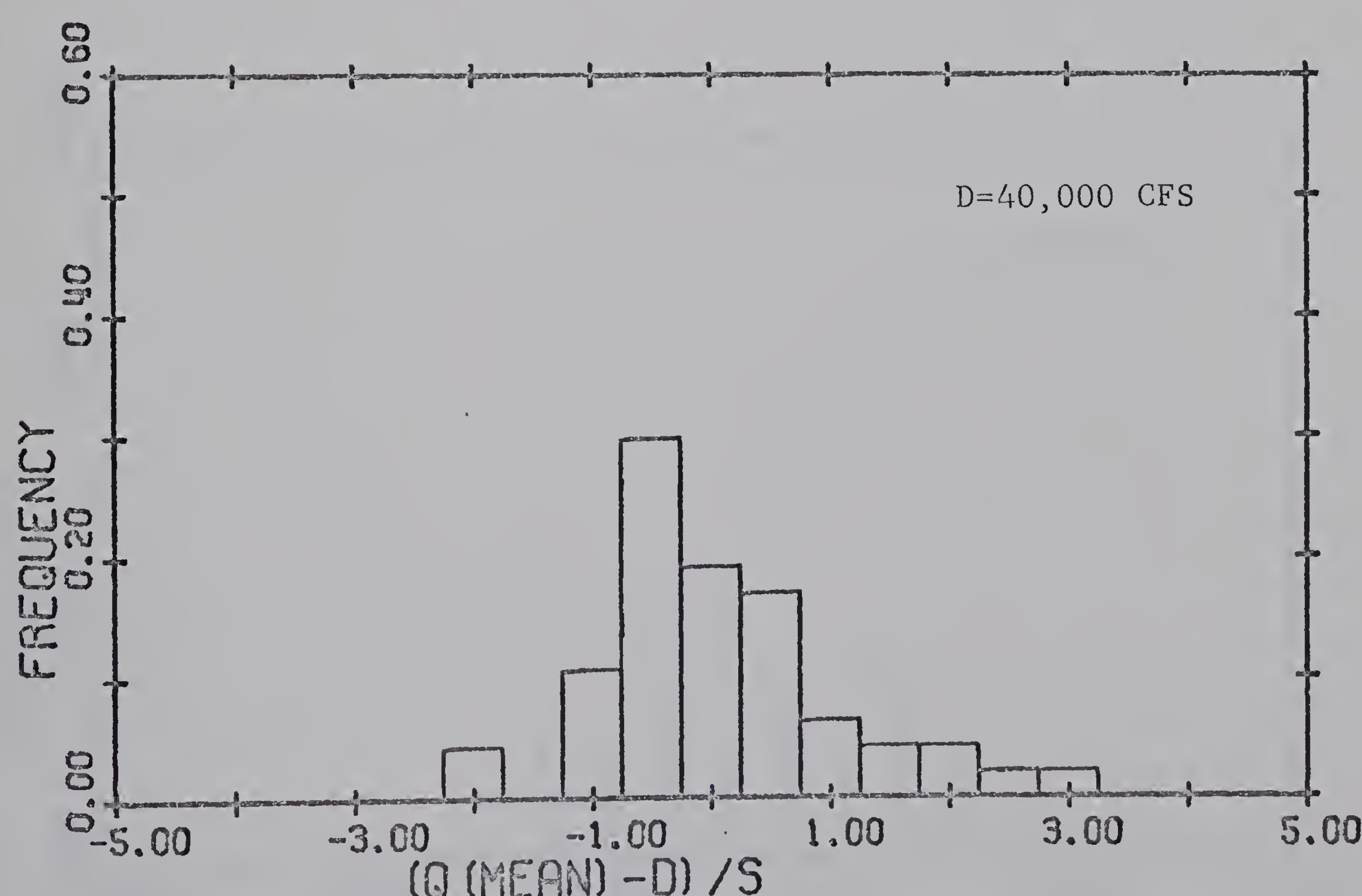
G. B-4A FREQUENCY HISTOGRAM OF Q (MEAN) :RISING CASES



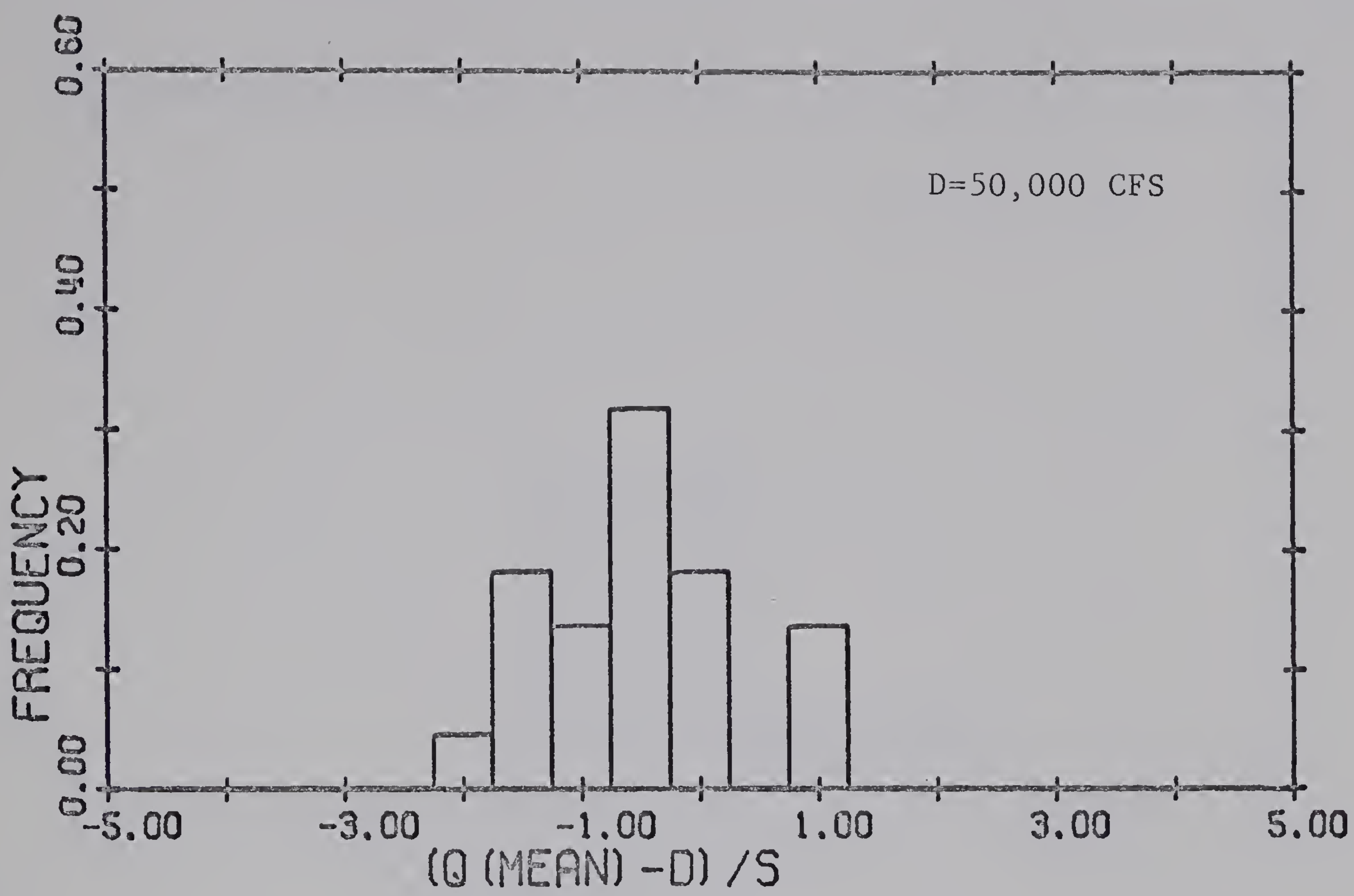
G. B-4B FREQUENCY HISTOGRAM OF Q (MEAN) :RECESSION CASES



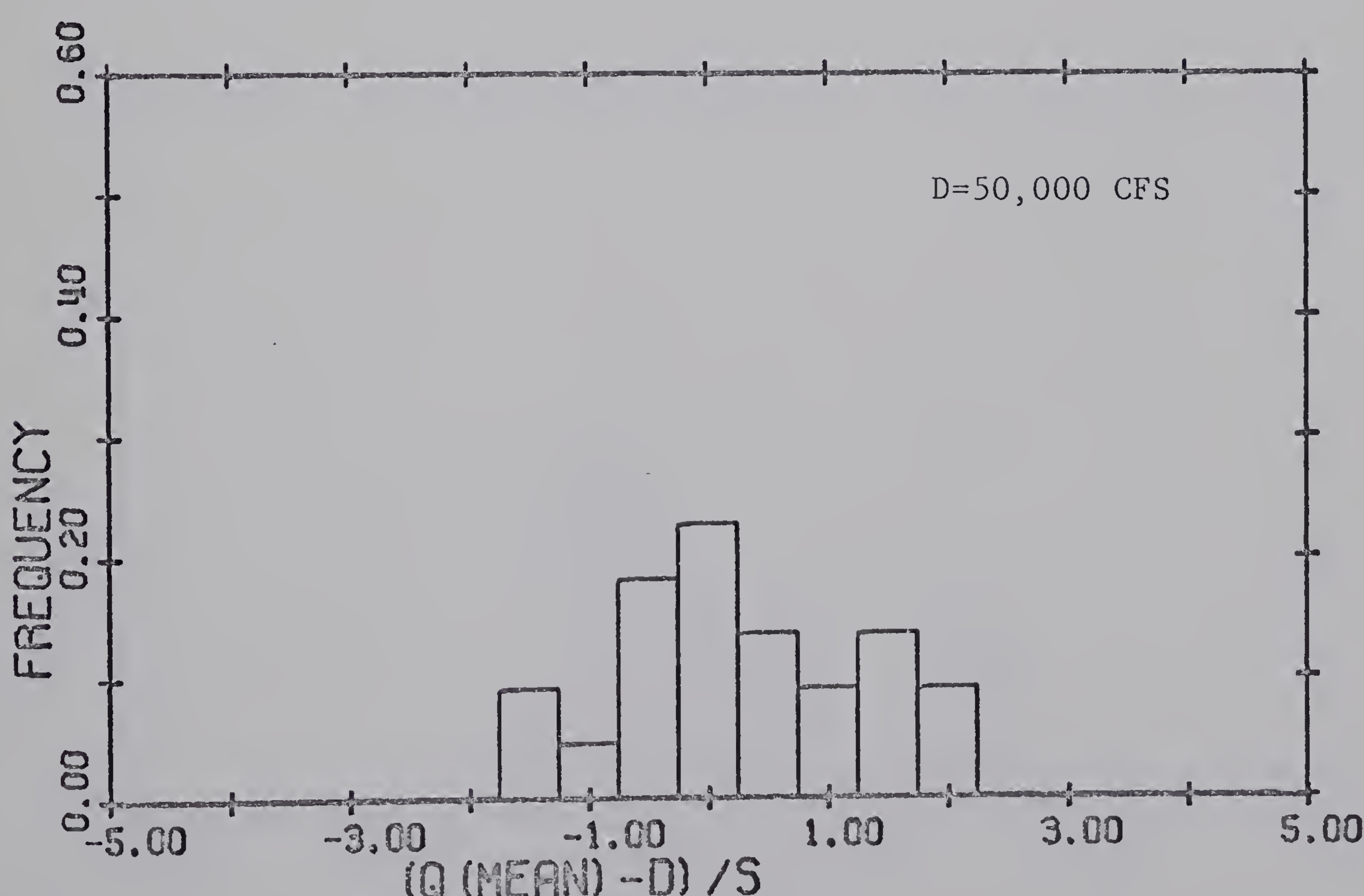
G. B-5A FREQUENCY HISTOGRAM OF Q (MEAN) : RISING CASES



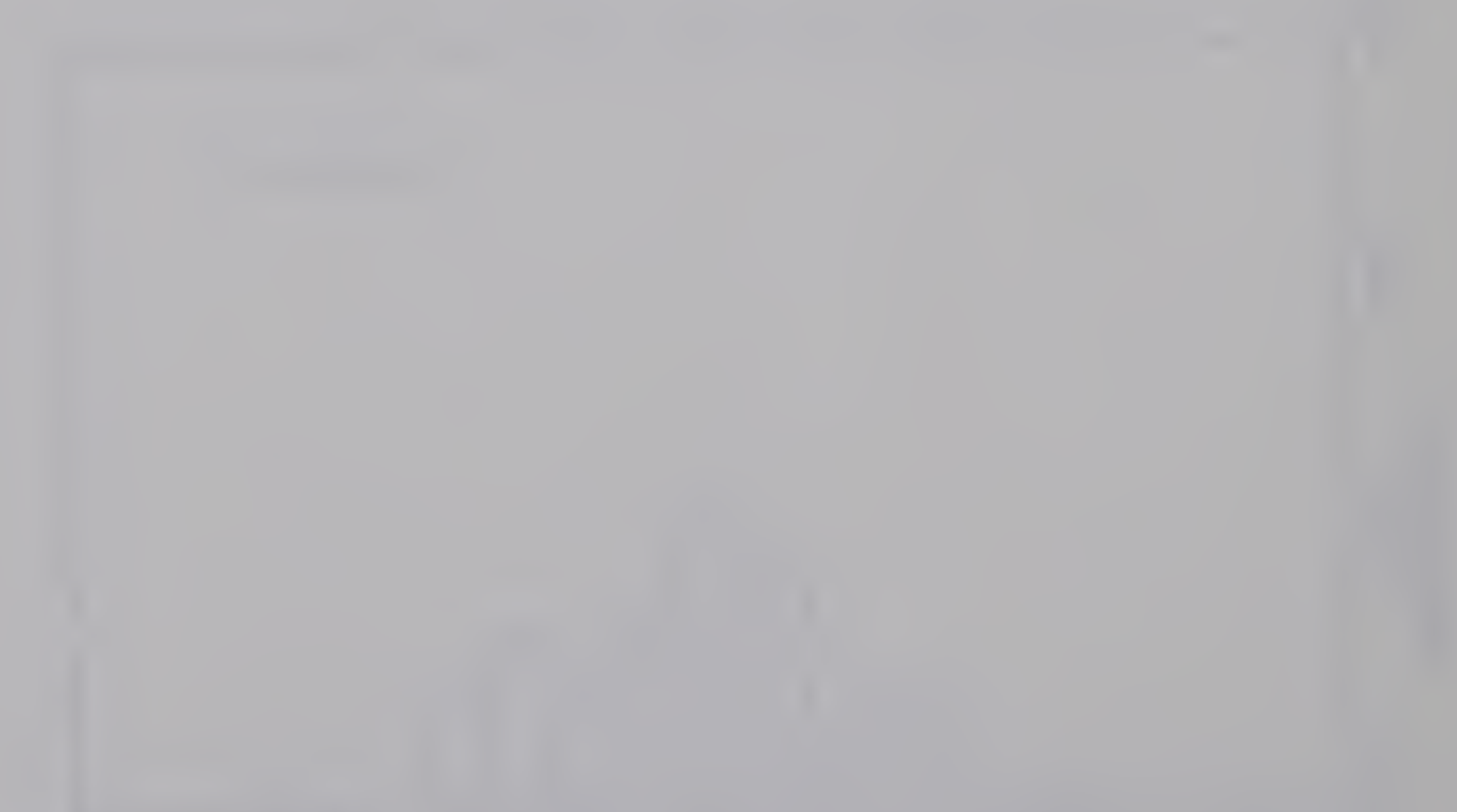
. B-5B FREQUENCY HISTOGRAM OF Q (MEAN) : RECESSION CASES

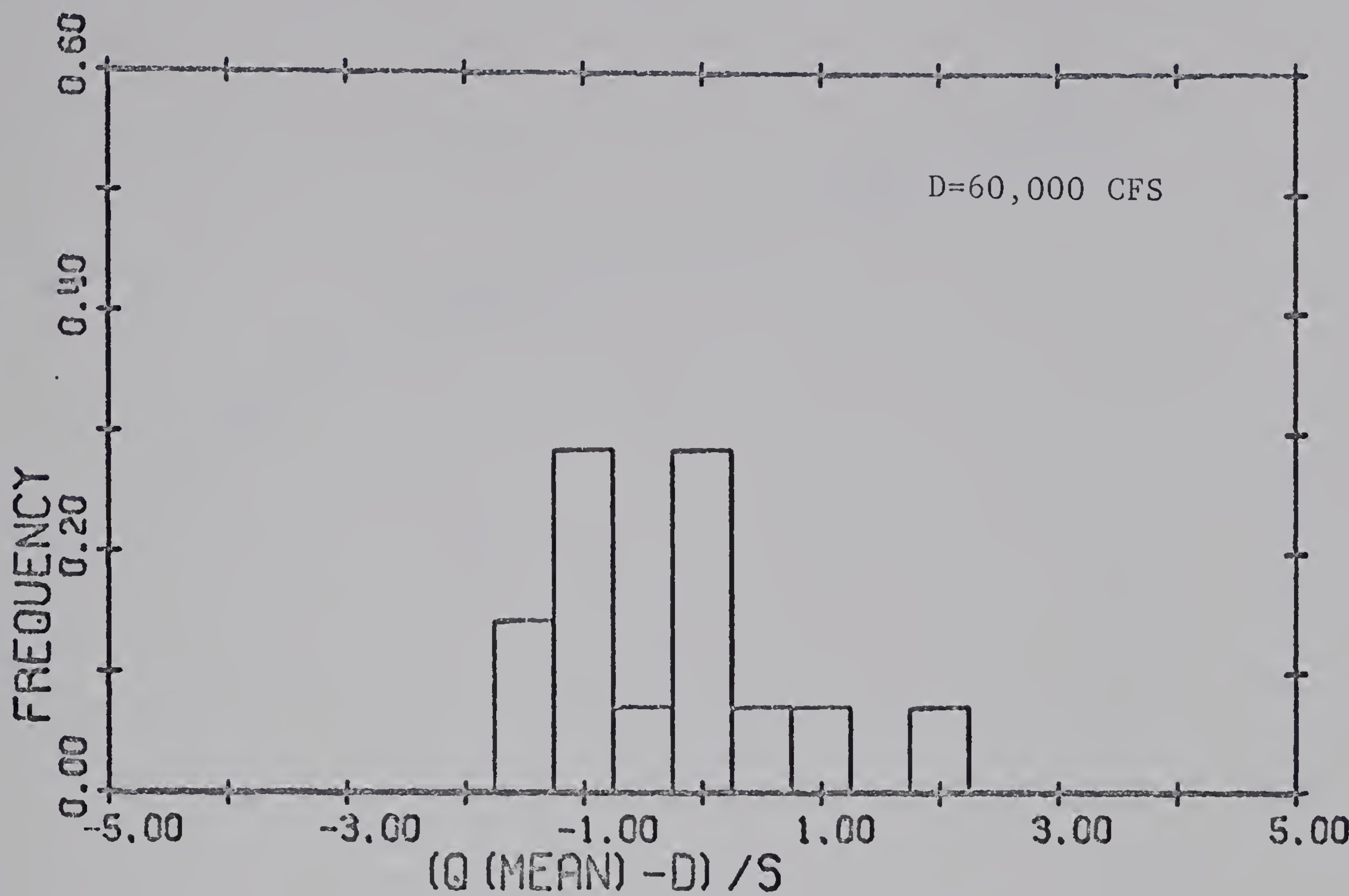


B-6A FREQUENCY HISTOGRAM OF Q (MEAN) : RISING CASES

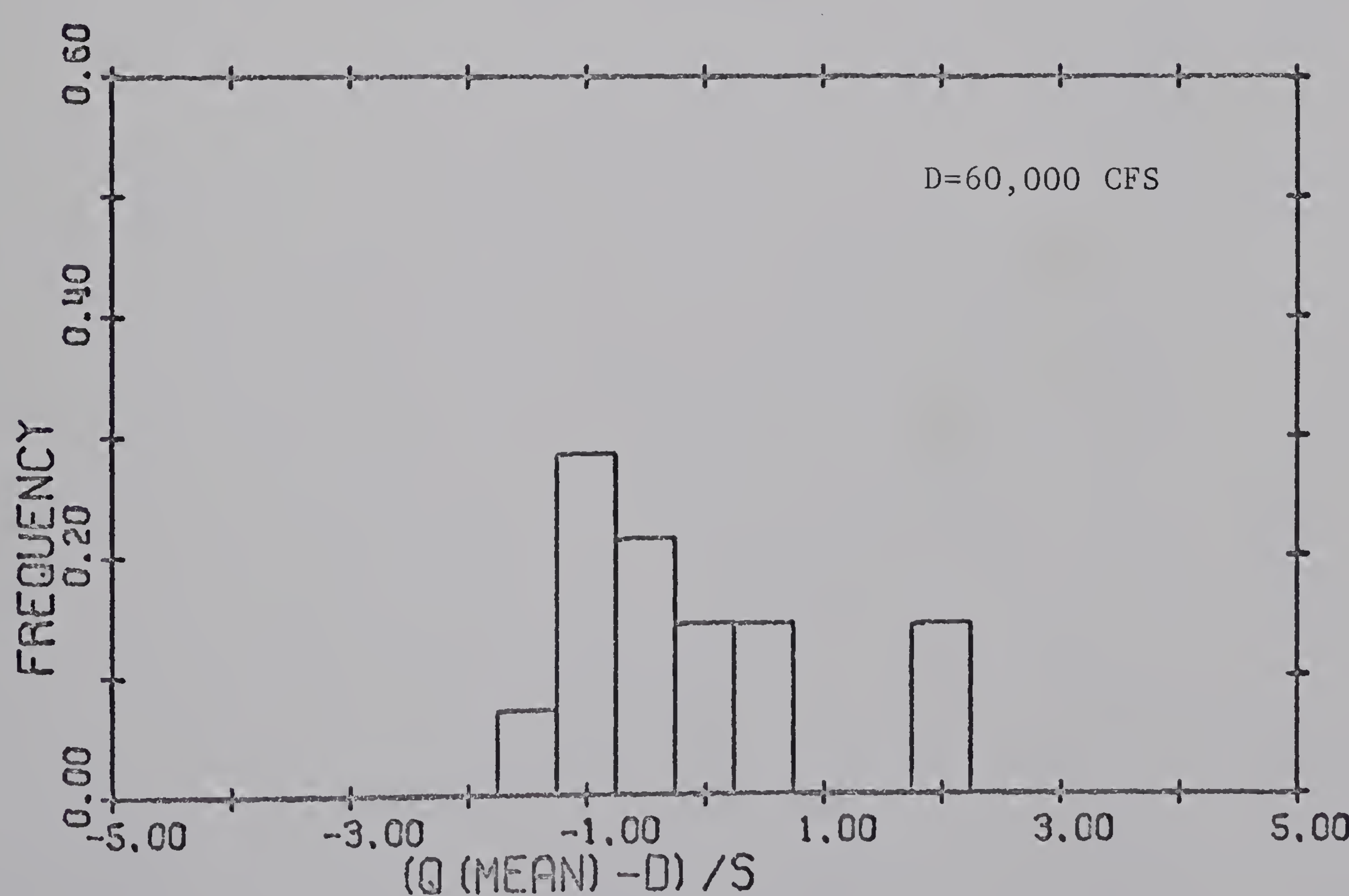


B-6B FREQUENCY HISTOGRAM OF Q (MEAN) : RECESSION CASES

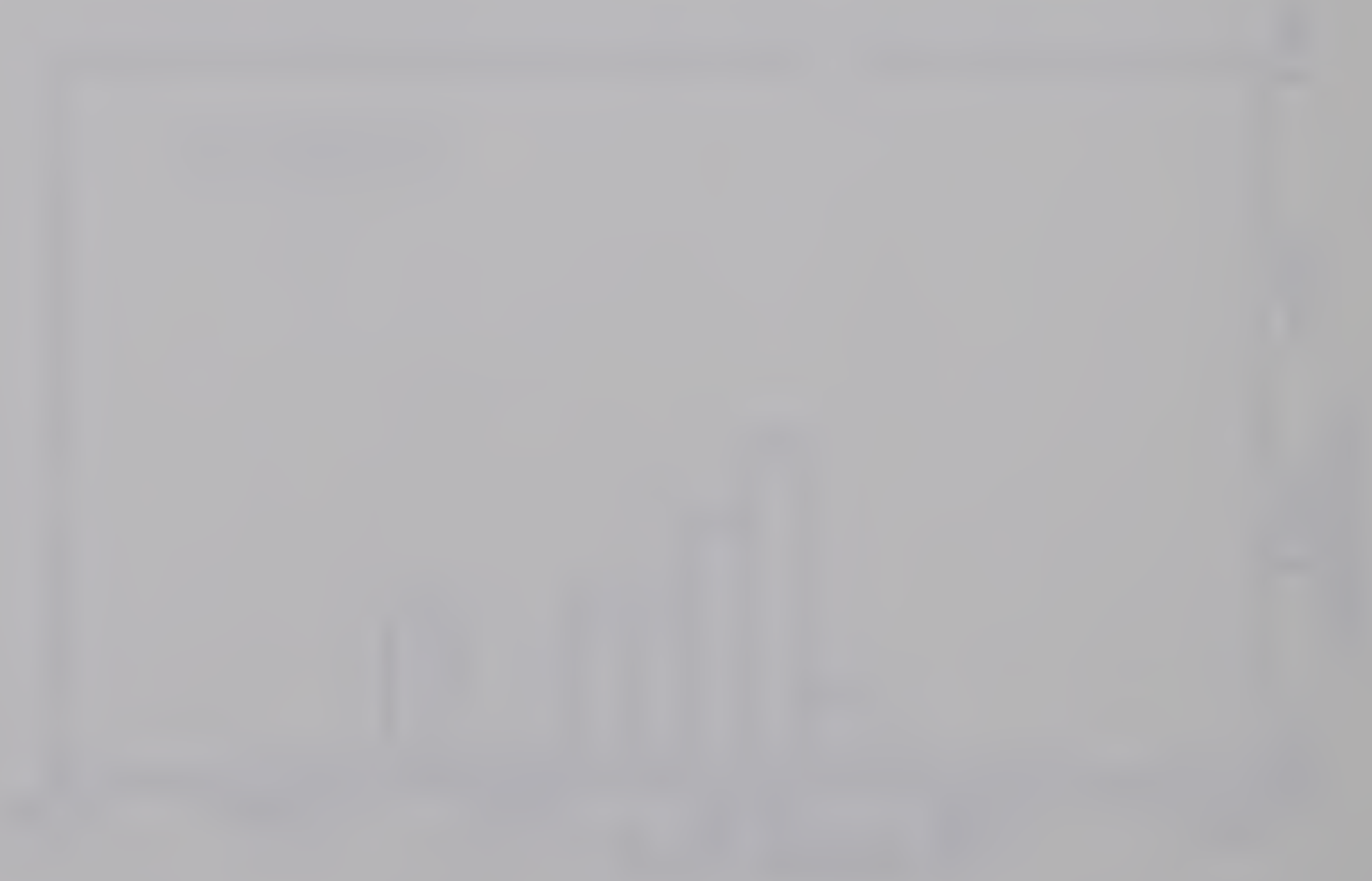




G. B-7A FREQUENCY HISTOGRAM OF Q (MEAN) :RISING CASES



G. B-7B FREQUENCY HISTOGRAM OF Q (MEAN) :RECESSION CASES



APPENDIX C

PROBABILITY CURVES OF Q_{RATIO}.

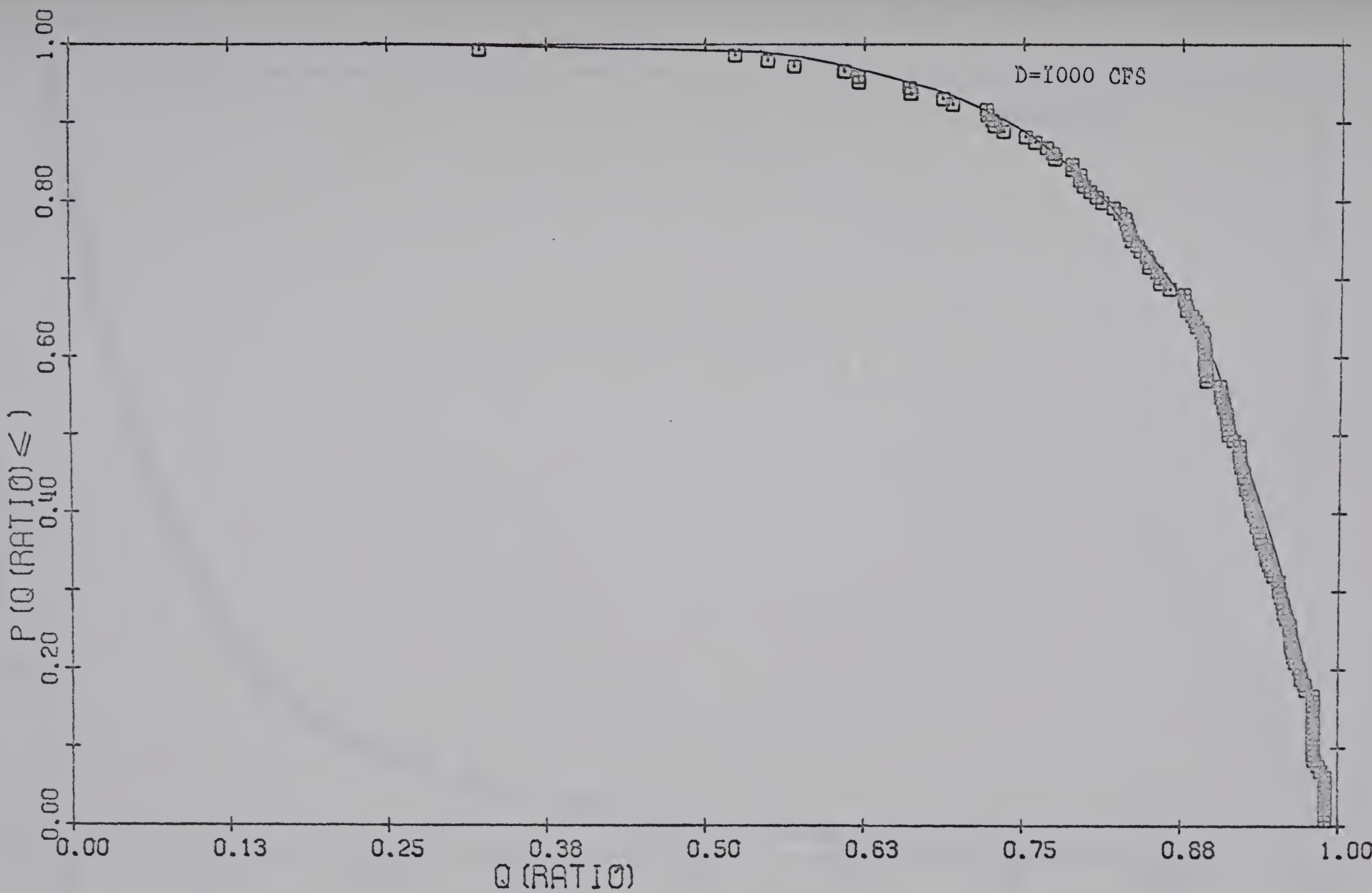


FIG. C-IA PRØBABILITY CURVE FØR Q(RATIØ): RISING CASES

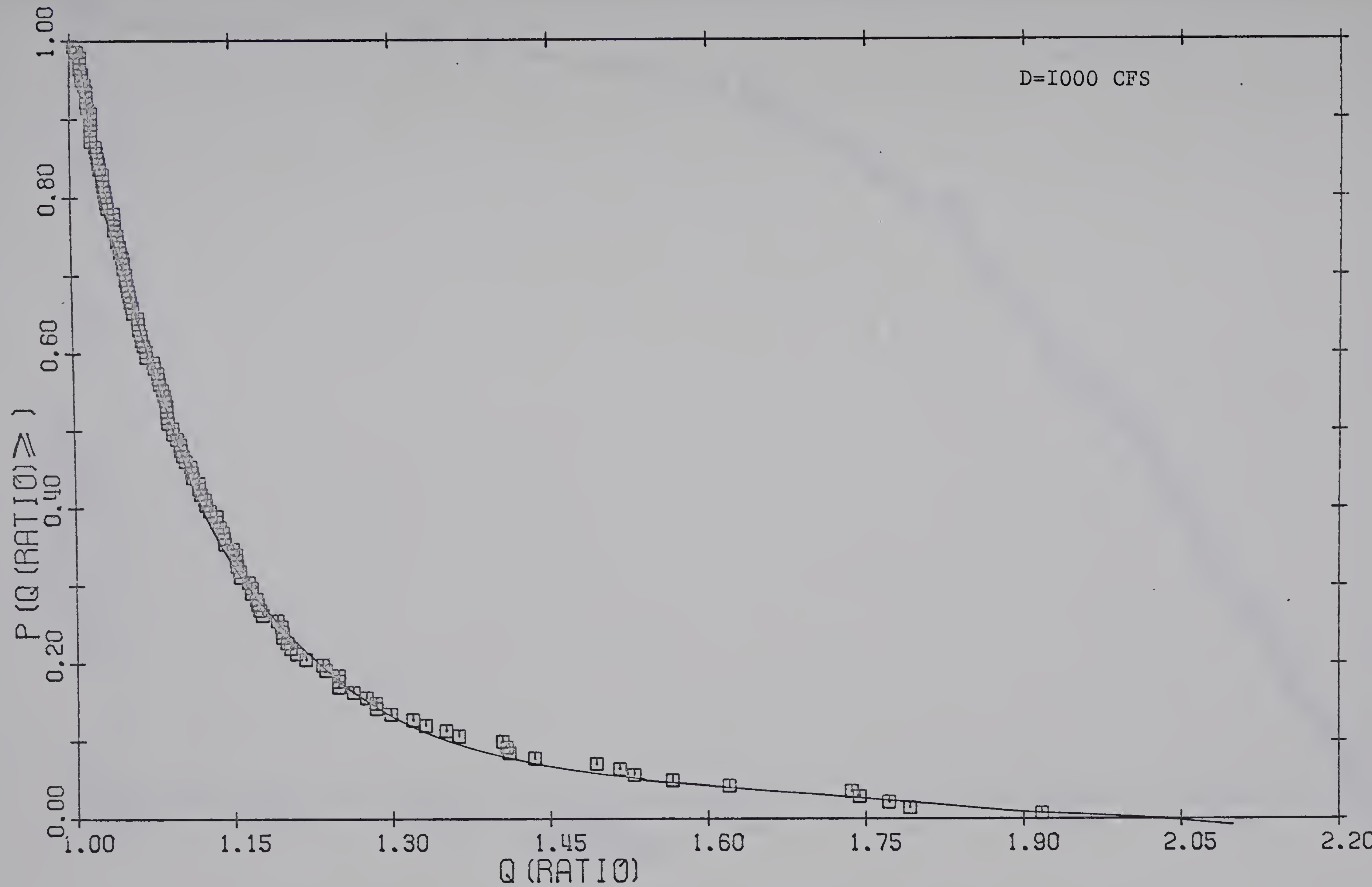


FIG. C-IB PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

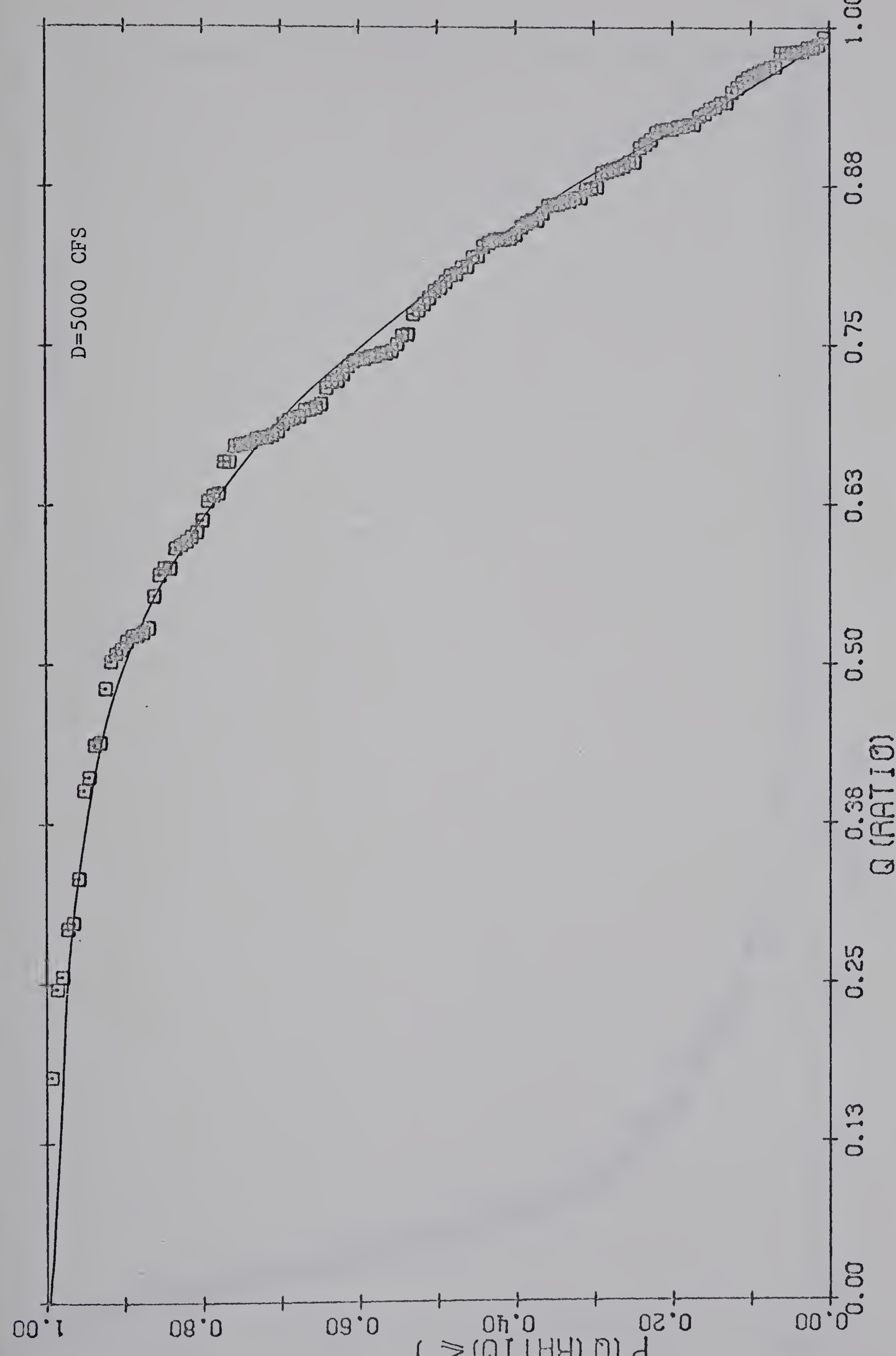


FIG. C-2A PROBABILITY CURVE FOR Q (RATIO) : RISING CASES

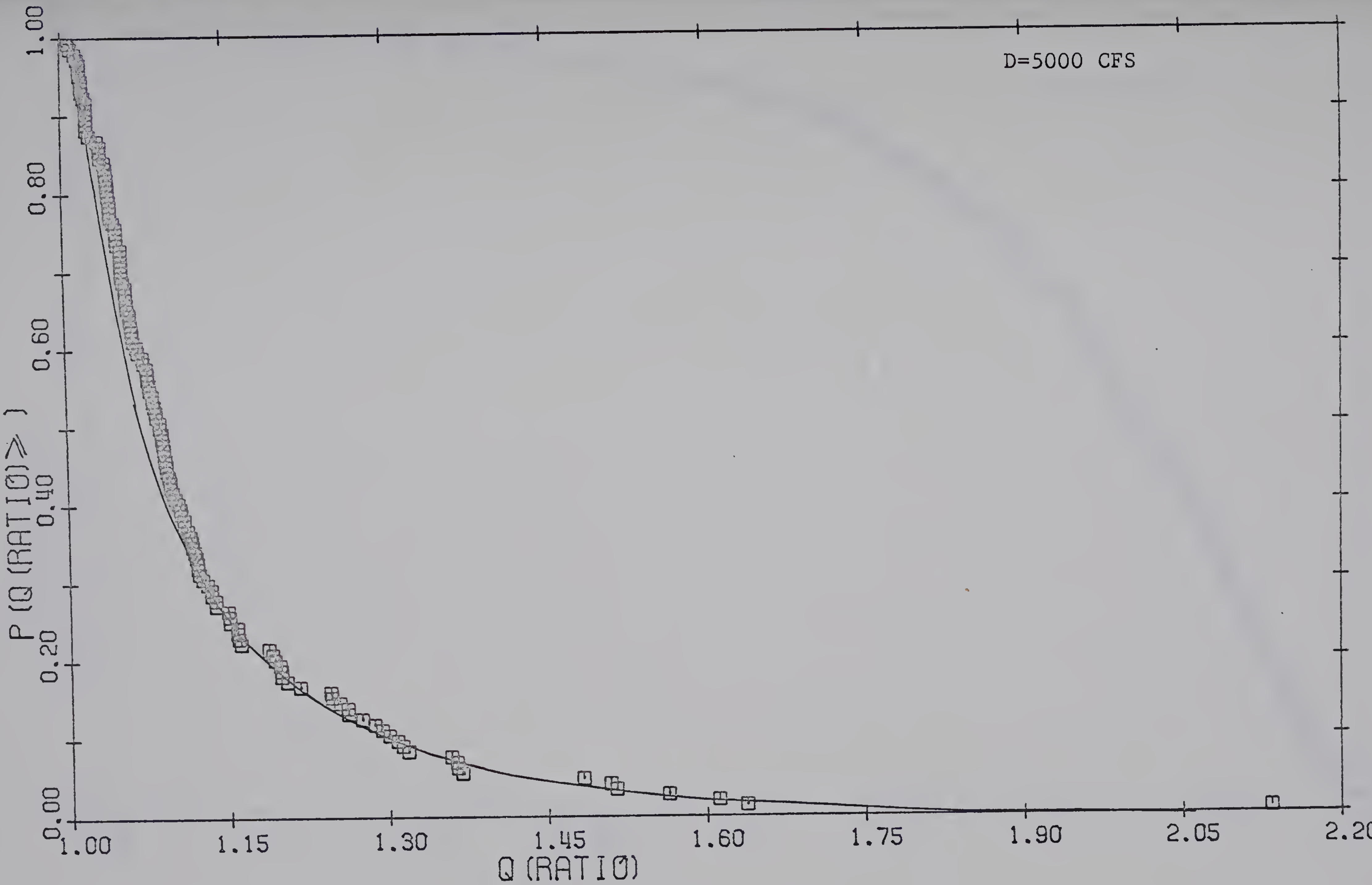


FIG. Ø-2B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

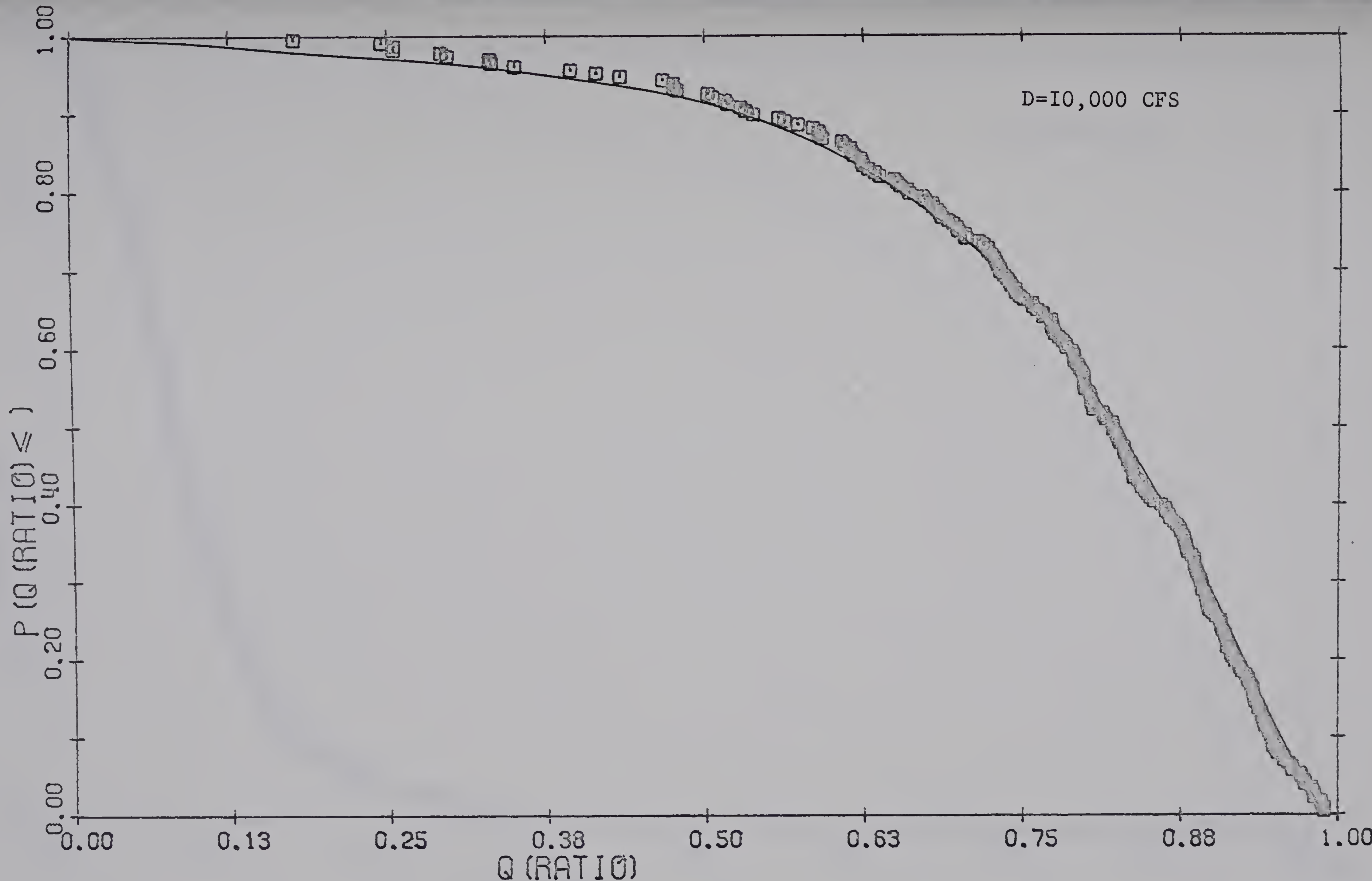


FIG. C-3A PRØBABILITY CURVE FØR Q (RATIØ): RISING CASES

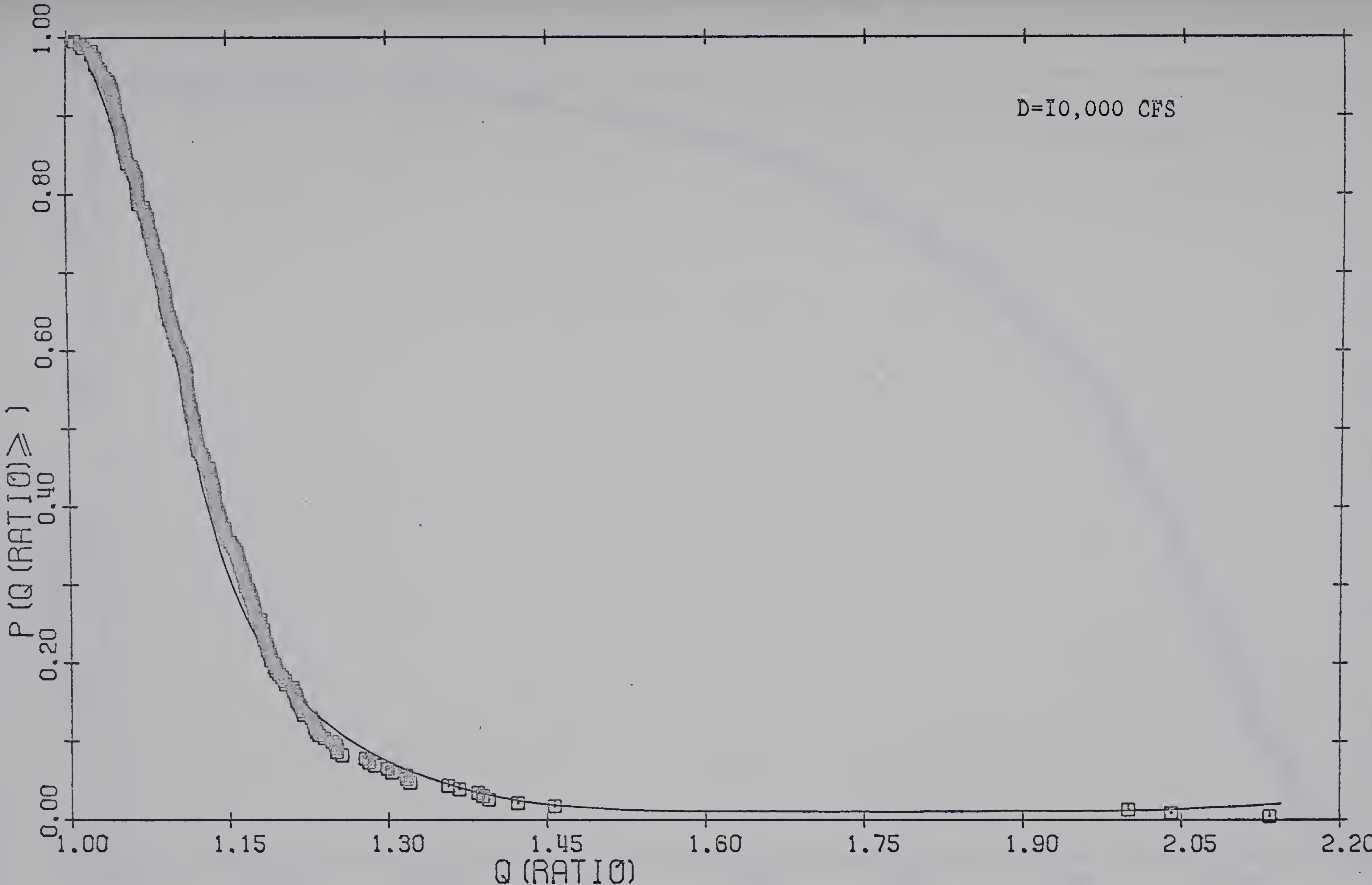


FIG. C-3B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

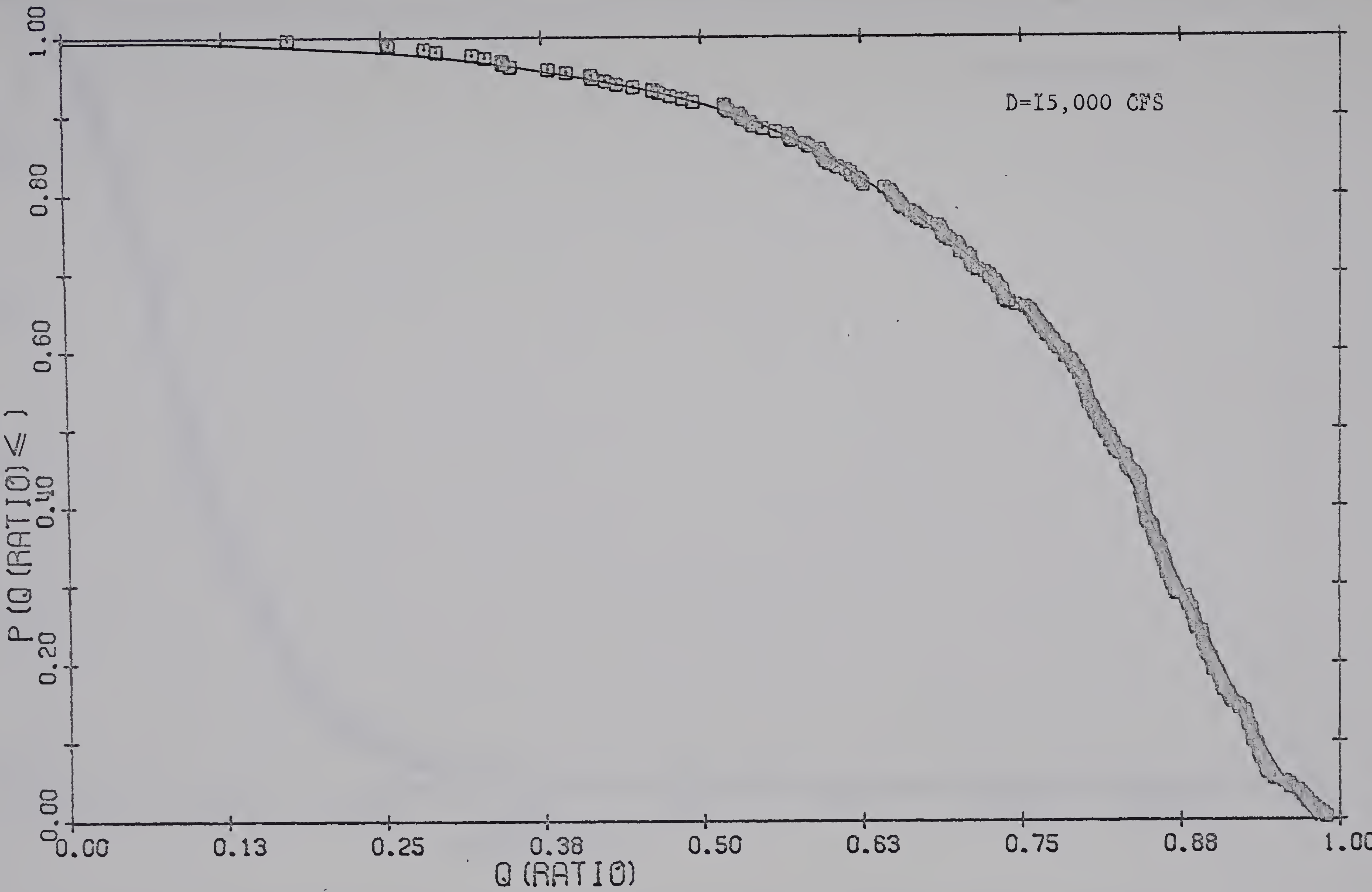


FIG. C-4A PROBABILITY CURVE FOR Q (RATIO): RISING CASES

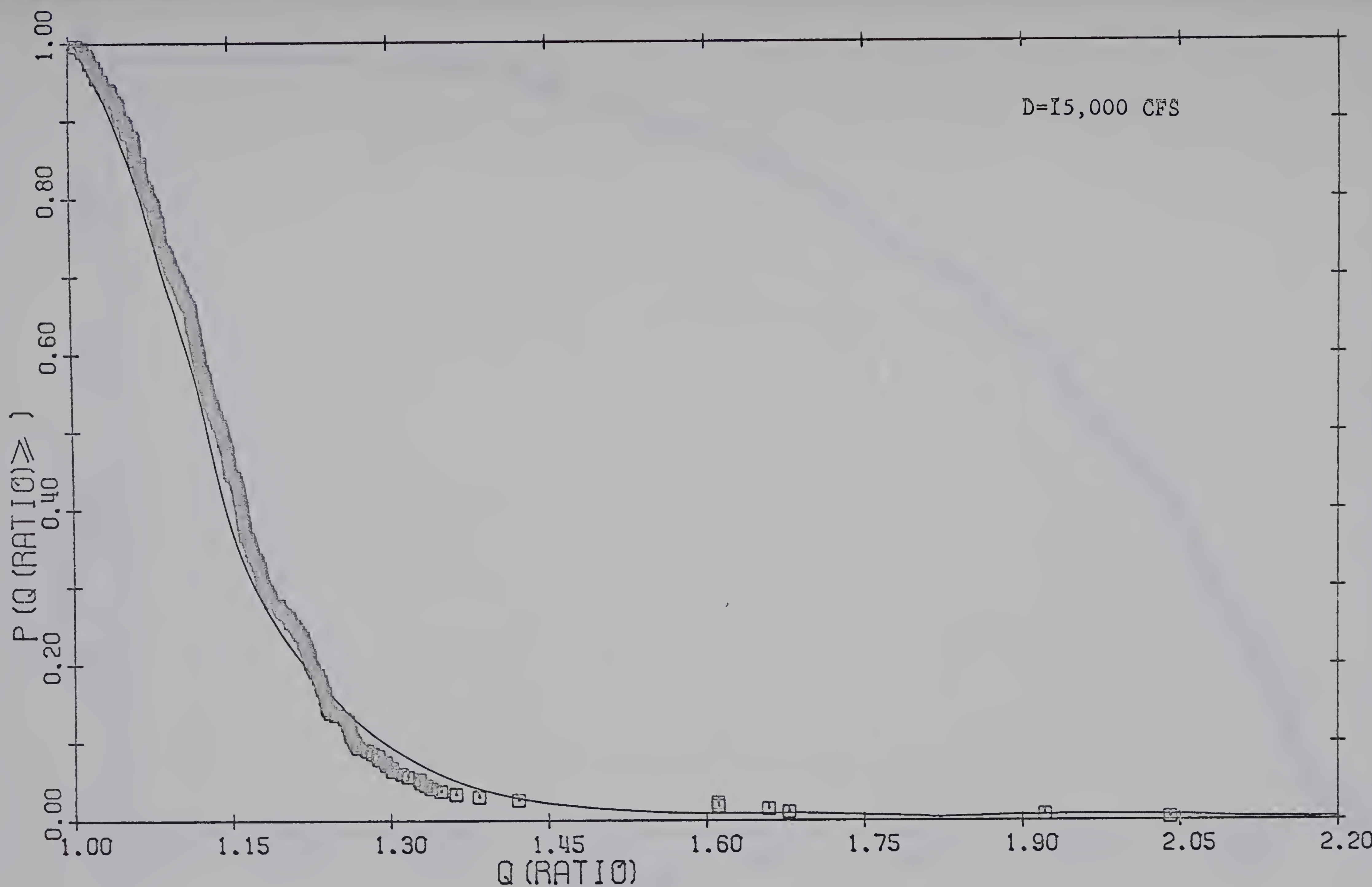


FIG. C-4B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

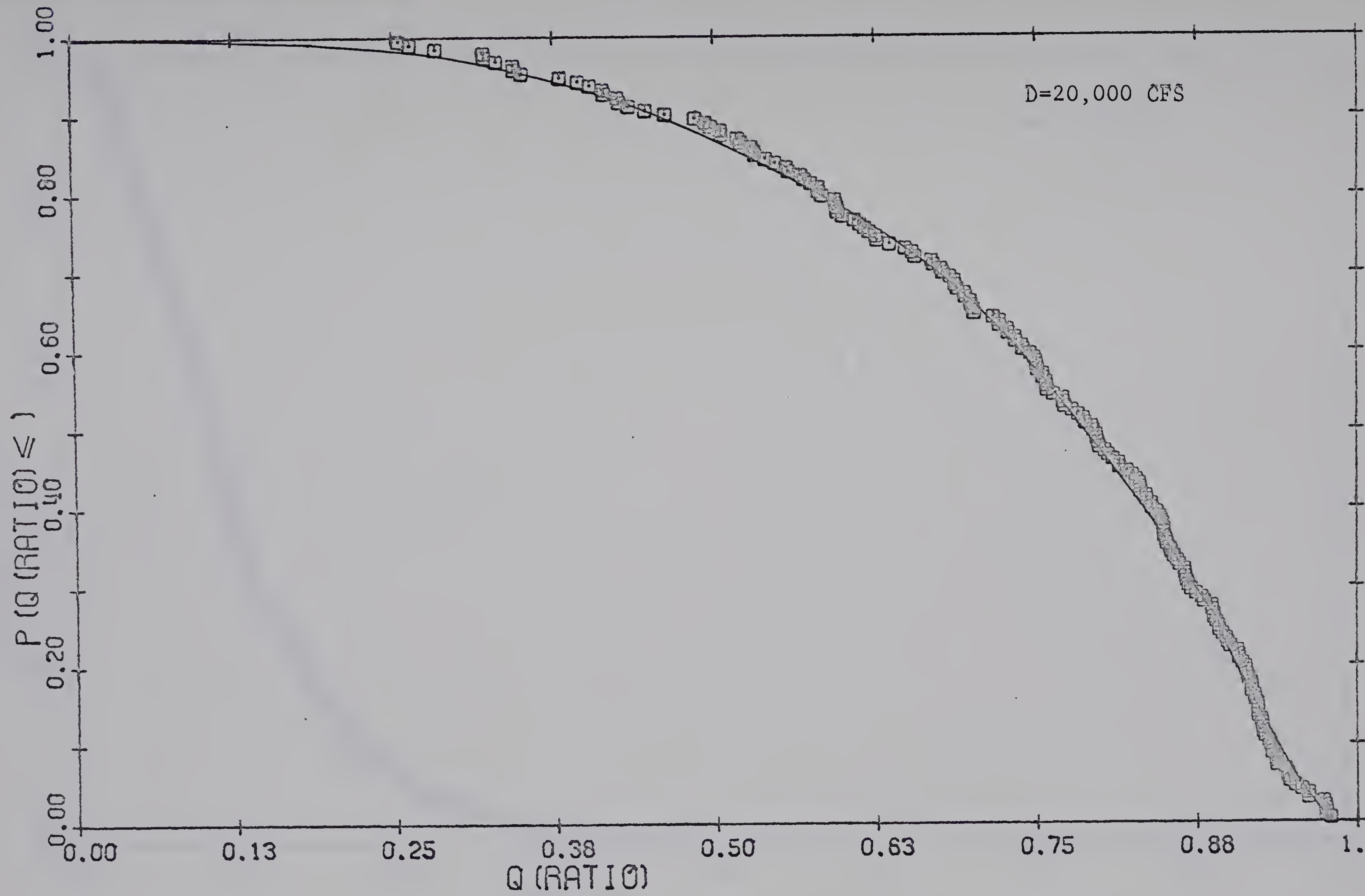


FIG. C-5A PRØBABILITY CURVE FØR Q (RATIØ) : RISING CASES

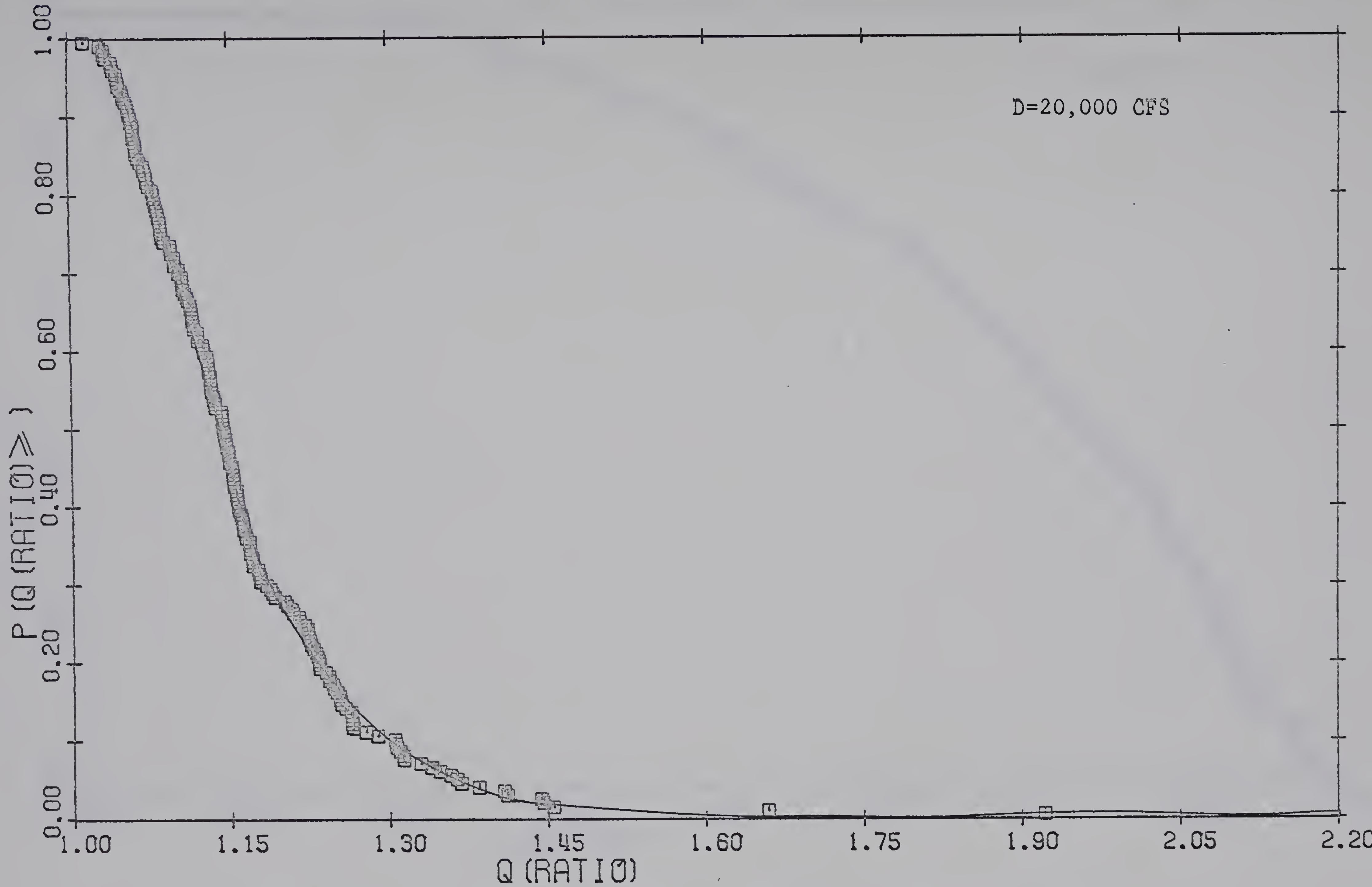


FIG. C-5B PRØBABILITY CURVE FØR Q (RATIØ): RECESSIØN CASES

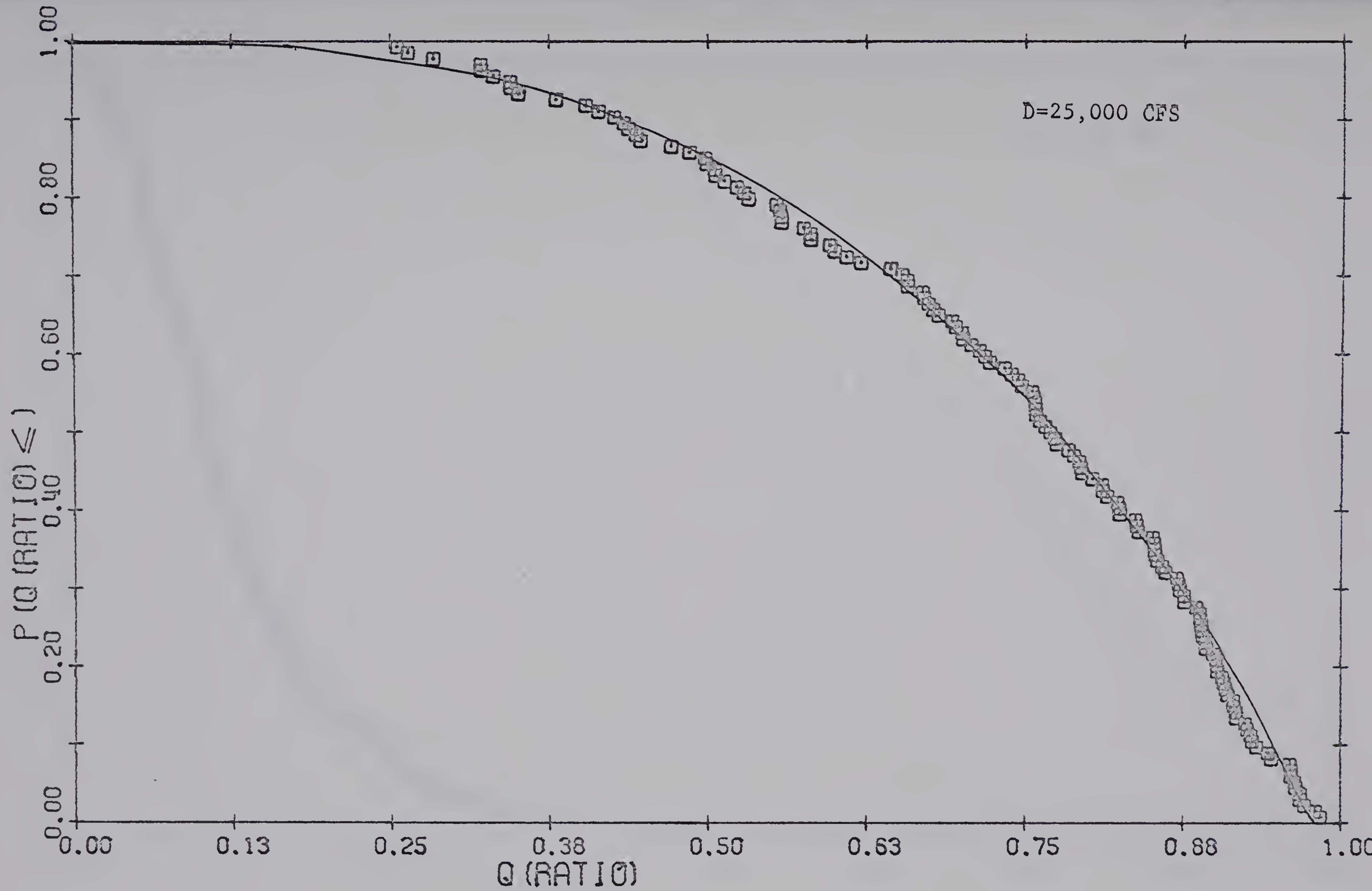


FIG. C-6A PROBABILITY CURVE FOR Q (RATIO): RISING CASES

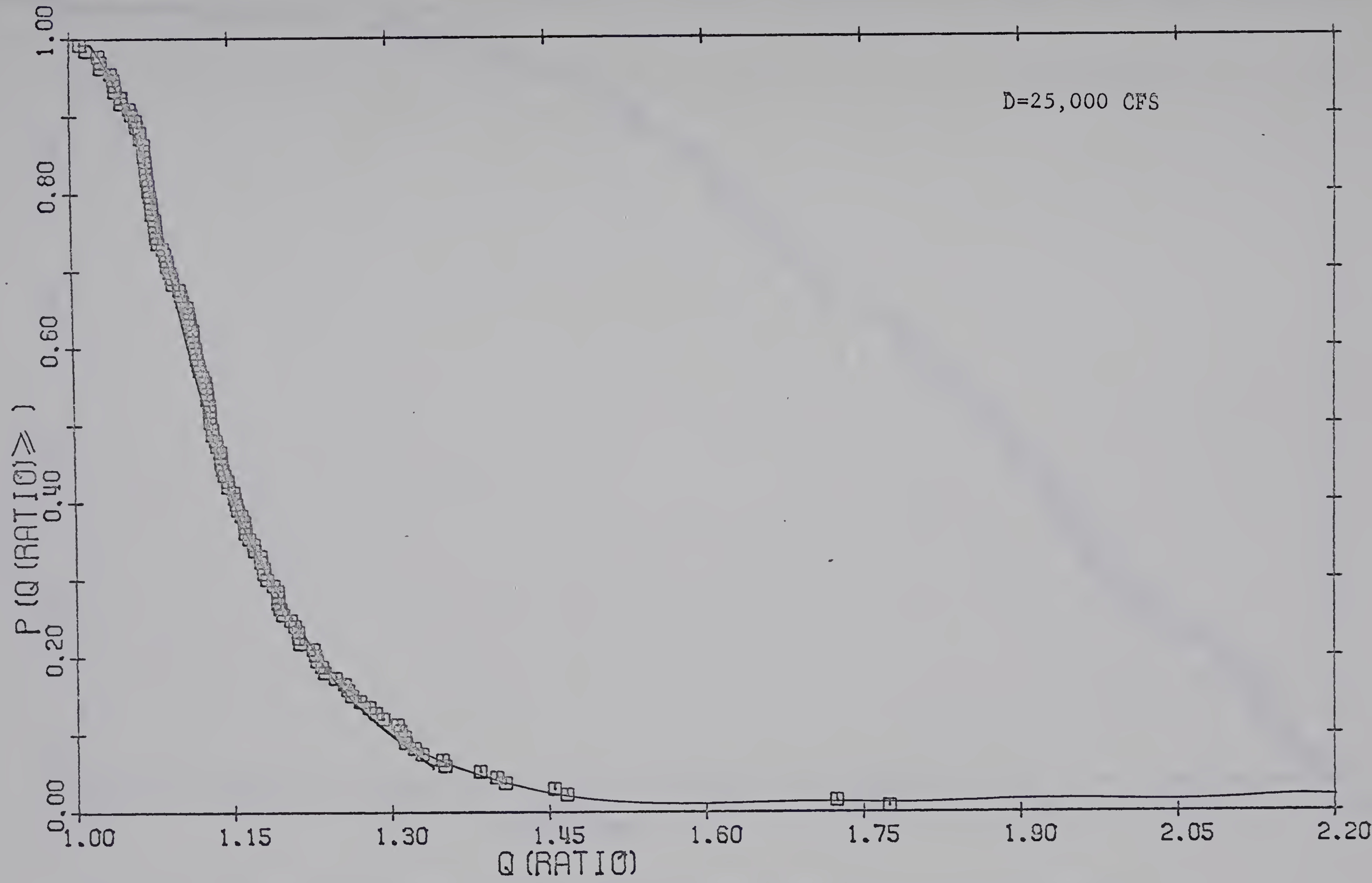


FIG. C-6B PROBABILITY CURVE FOR Q (RATIO): RECESSION CASES

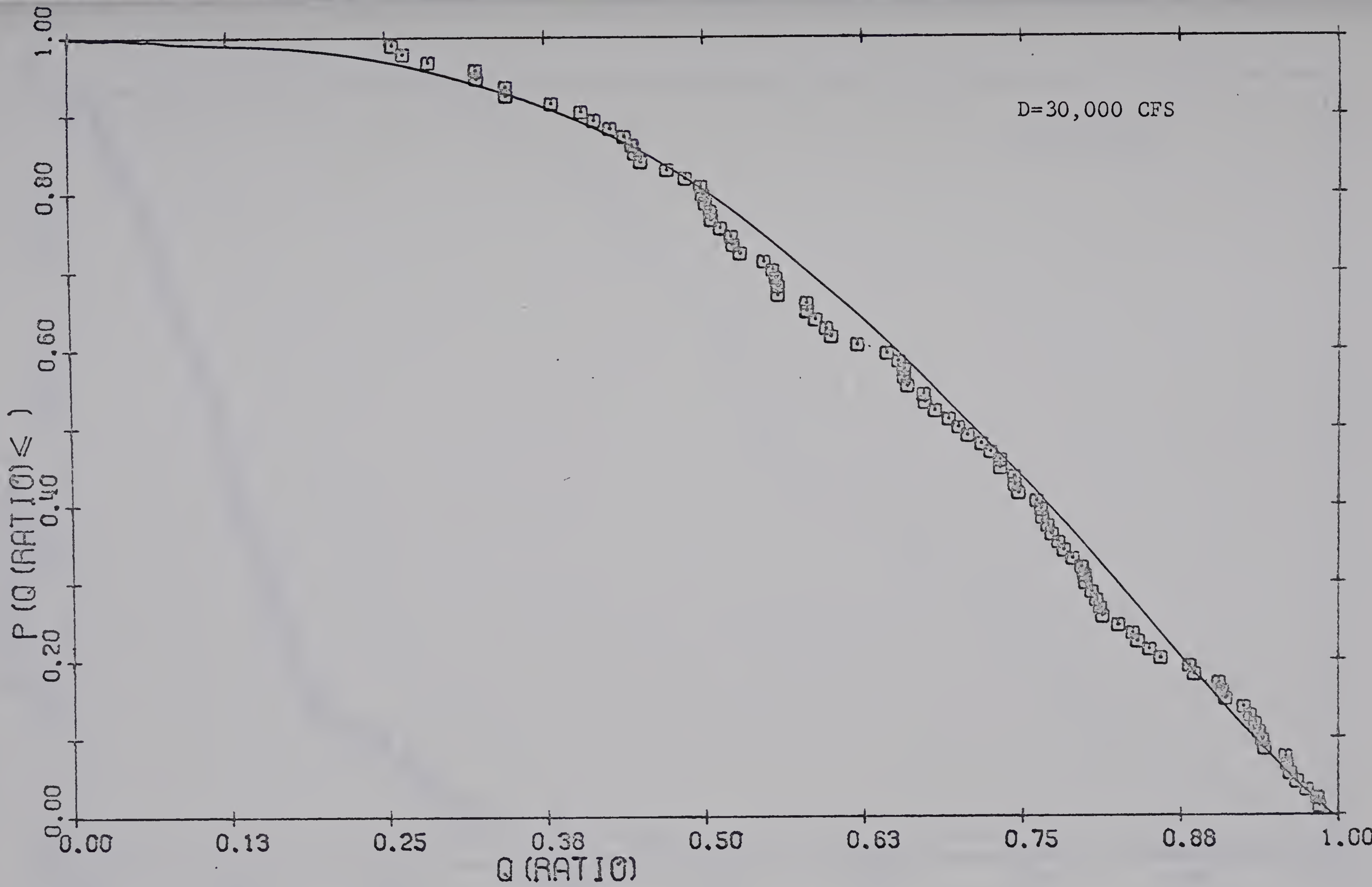


FIG. C-7A PROBABILITY CURVE FOR Q (RATIO): RISING CASES

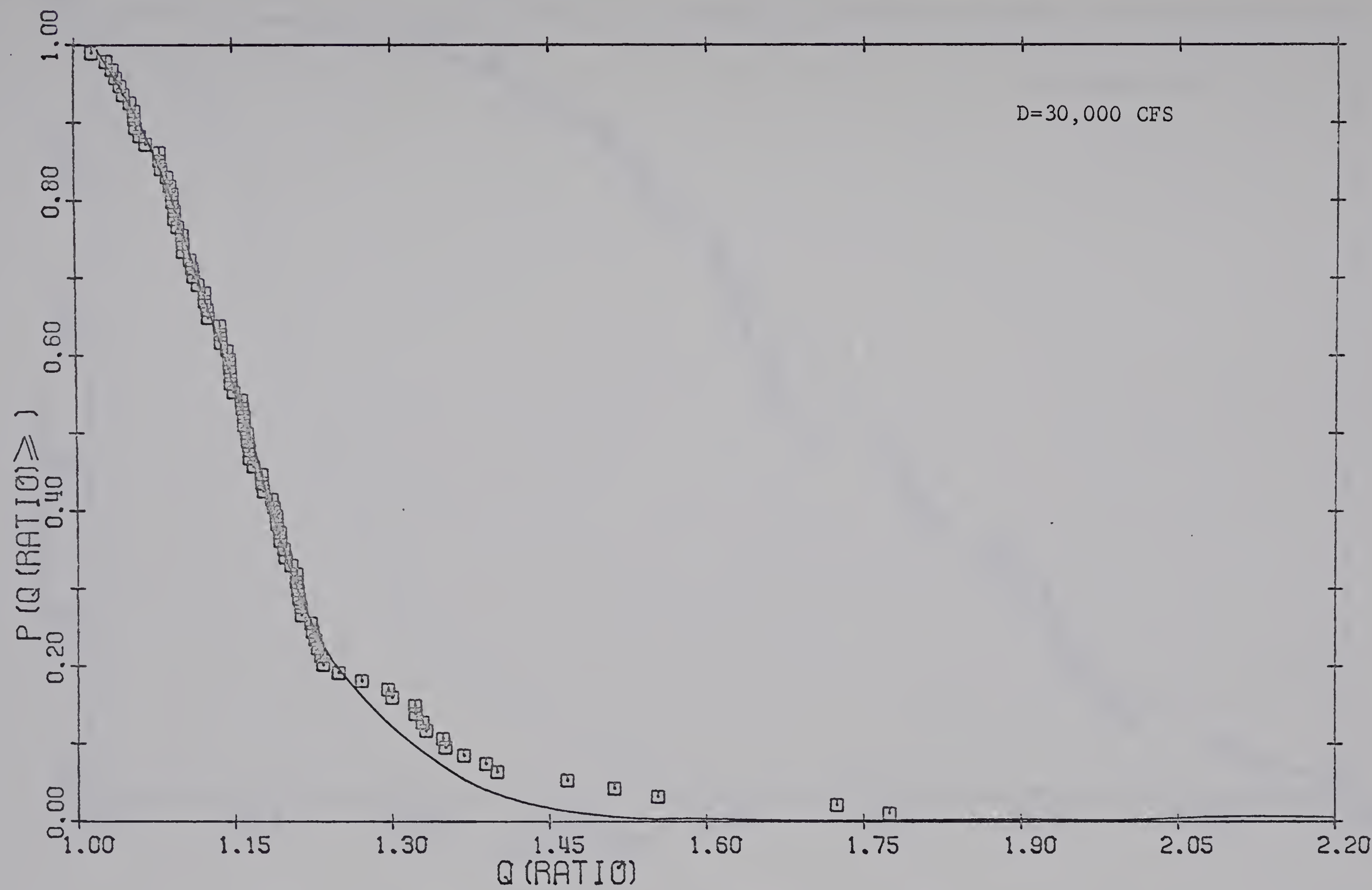


FIG. C-7B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

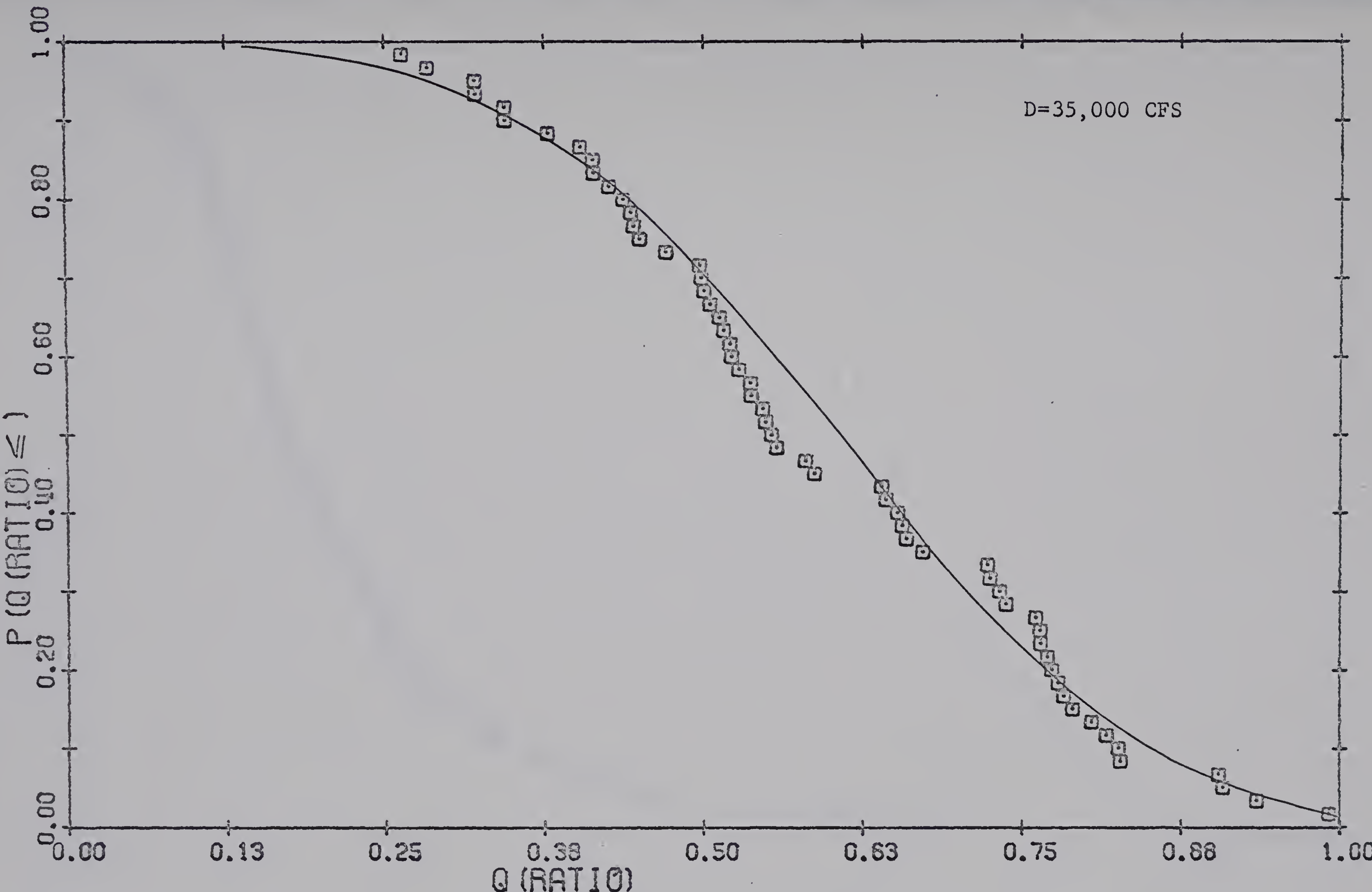


FIG. C-8A PROBABILITY CURVE FOR Q (RATIO): RISING CASES

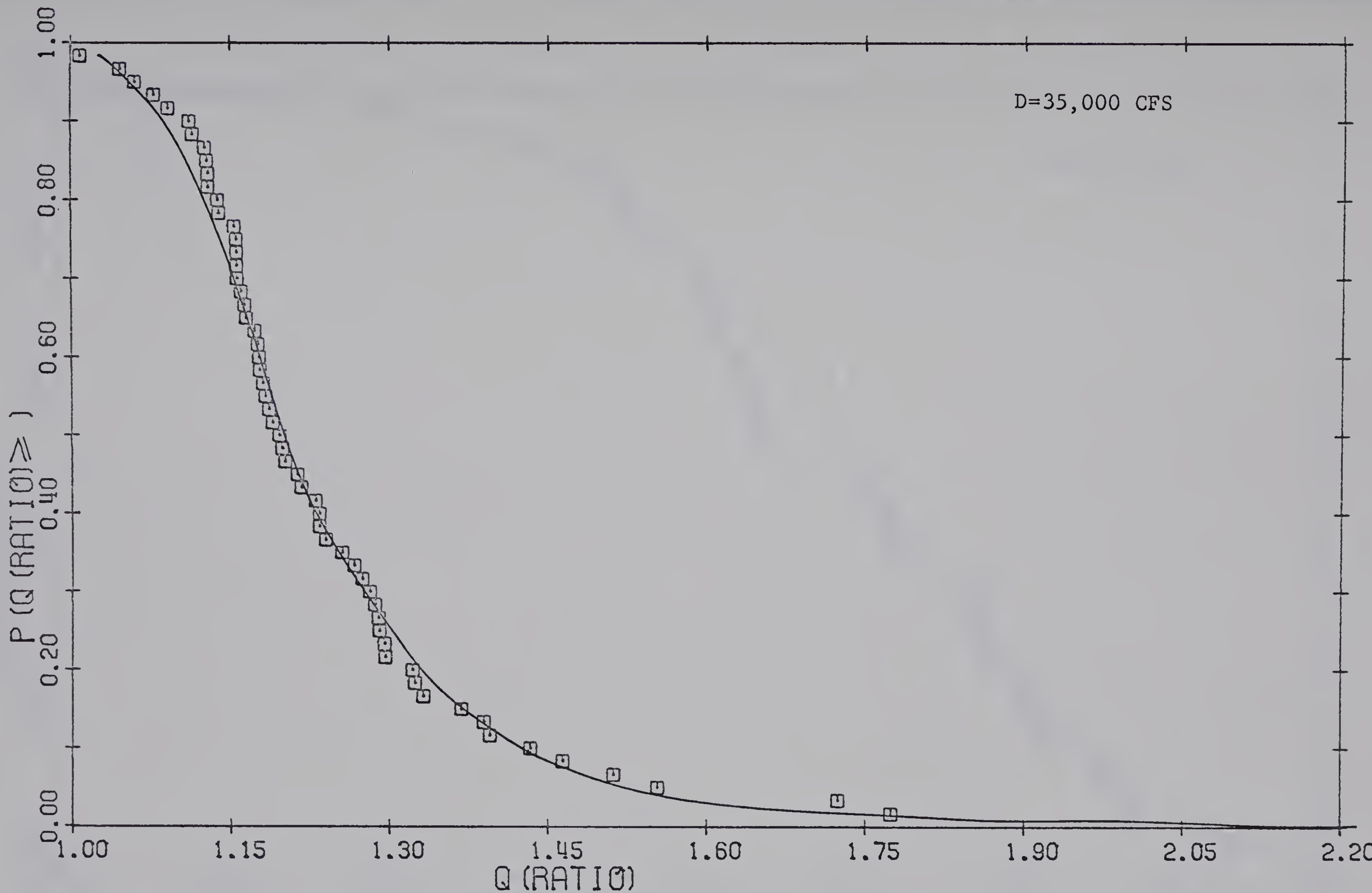


FIG. C-8B PROBABILITY CURVE FOR Q (RATIO): RECESSION CASES

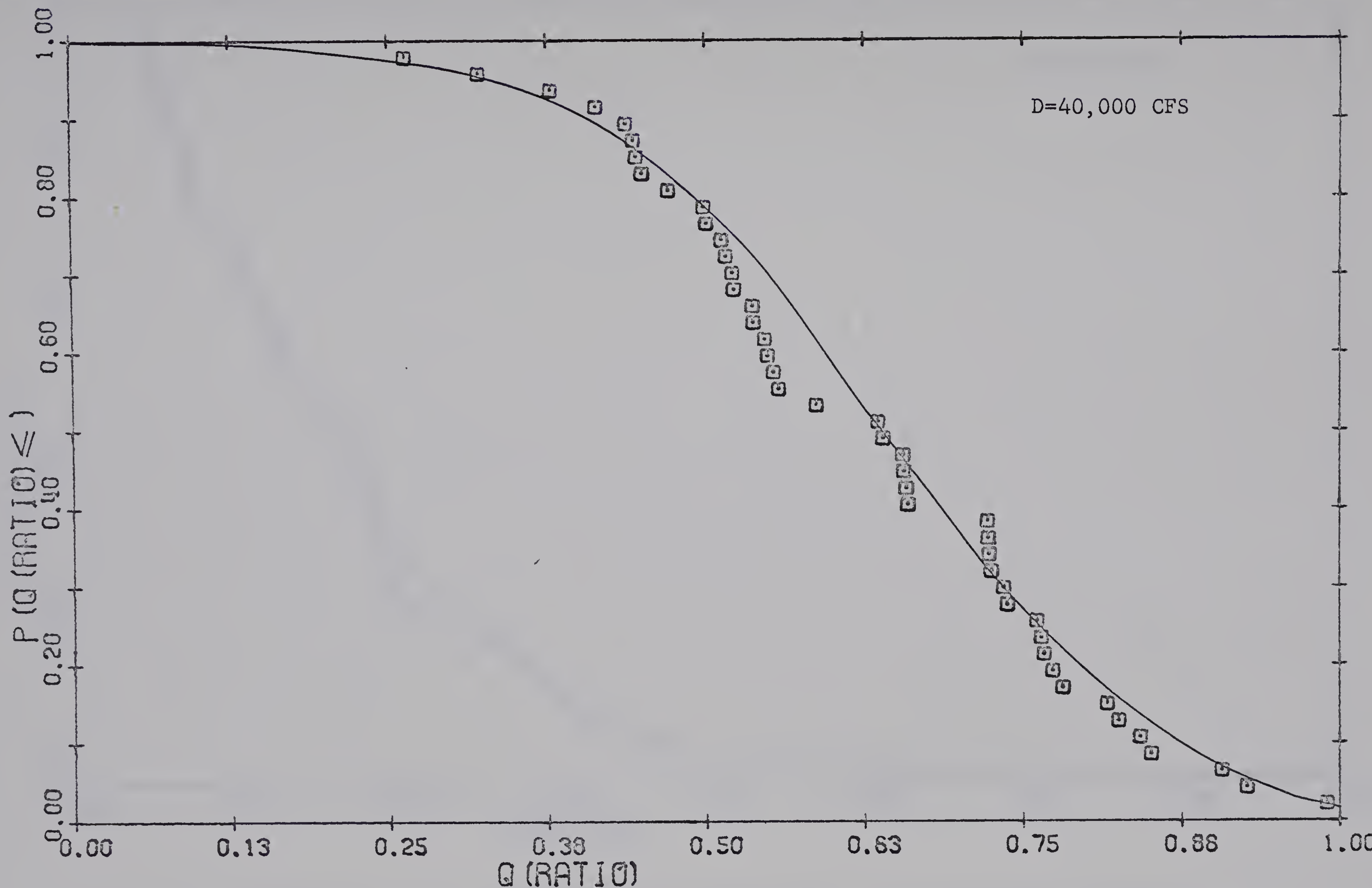


FIG. C-9A PROBABILITY CURVE FOR Q (RATIO): RISING CASES

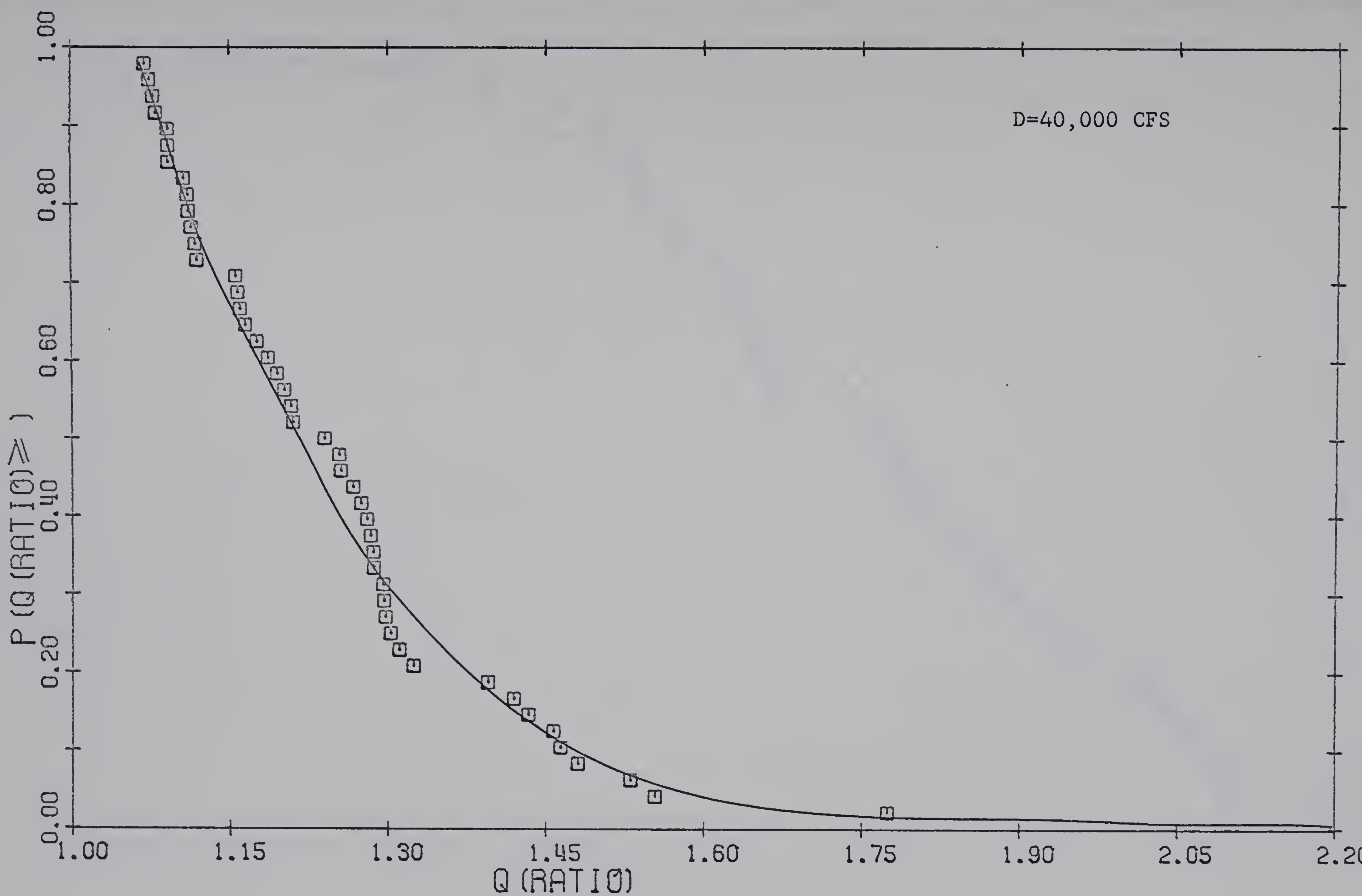


FIG. C-9B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

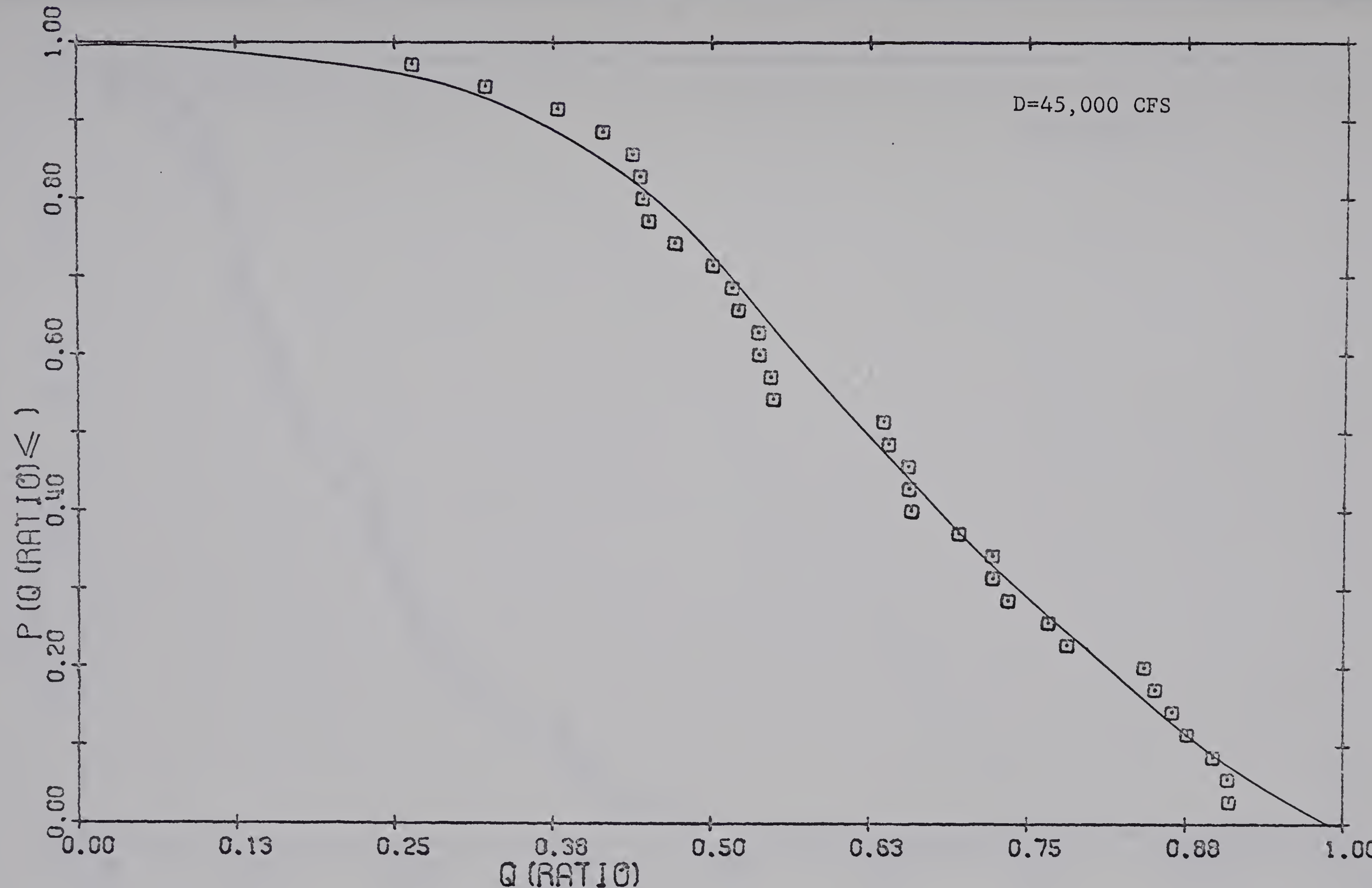


FIG. C-IOA PROBABILITY CURVE FOR Q (RATIO): RISING CASES

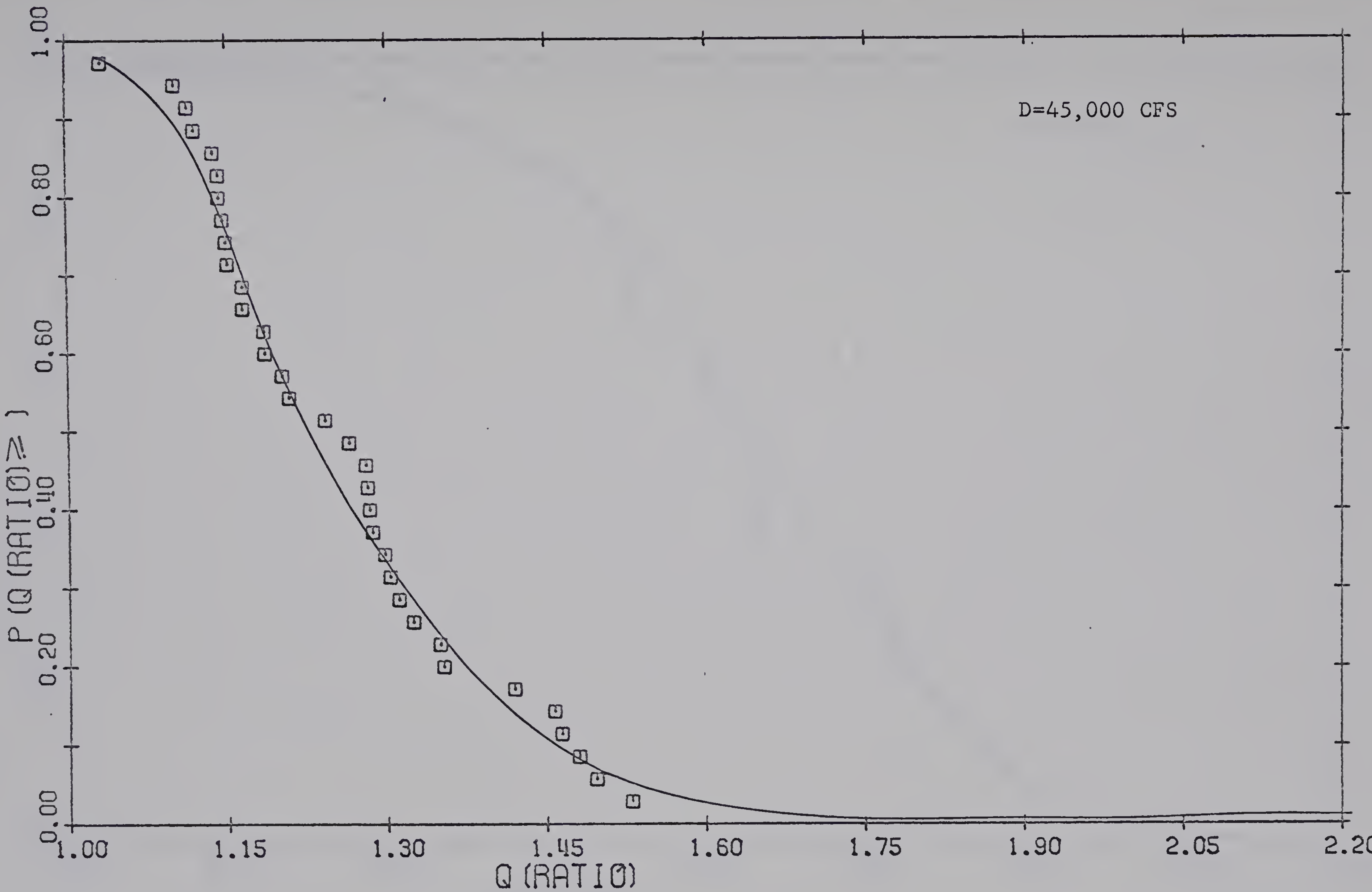


FIG. C-10B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

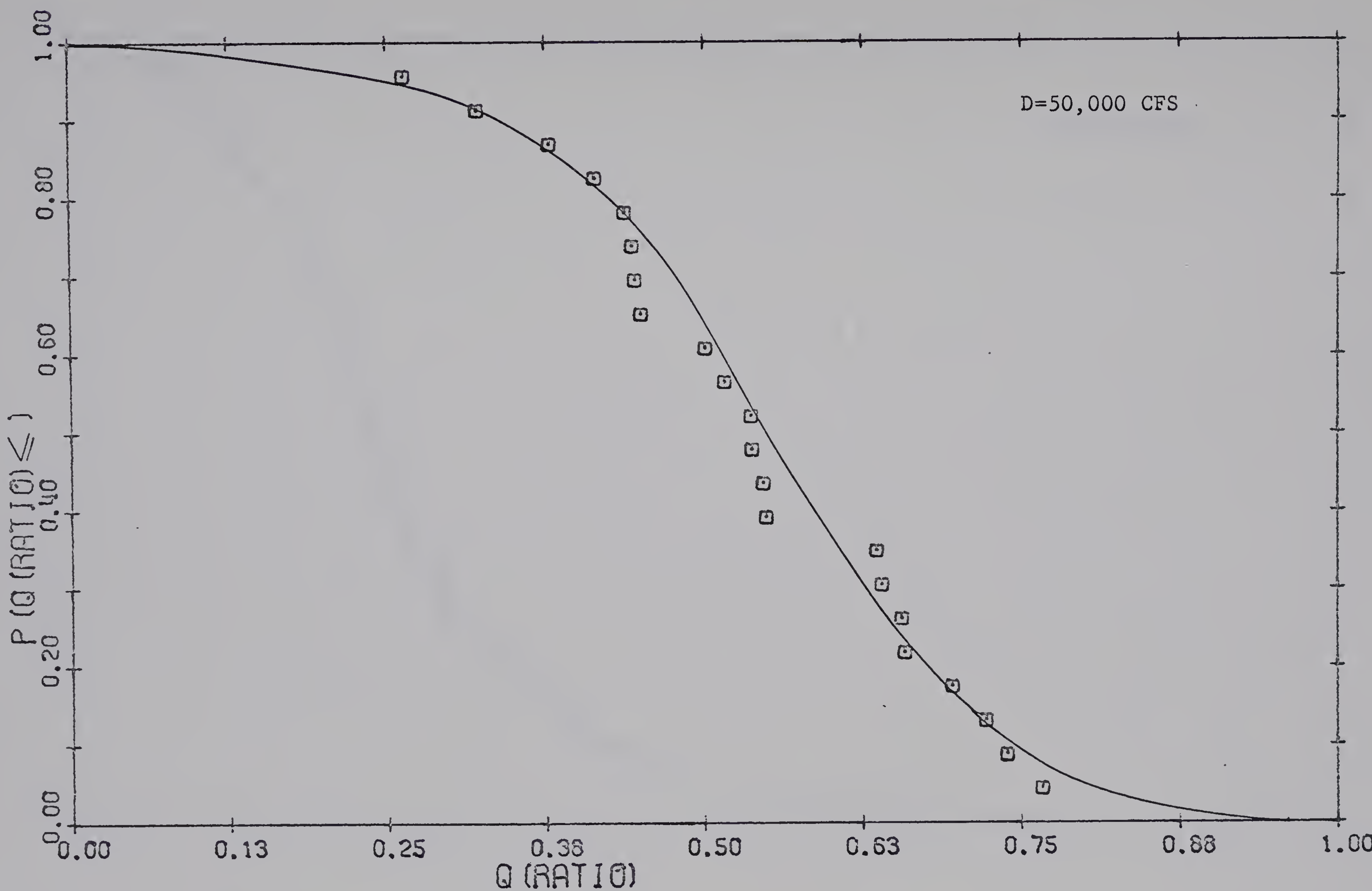


FIG. C-IIA PROBABILITY CURVE FOR Q (RATIO): RISING CASES

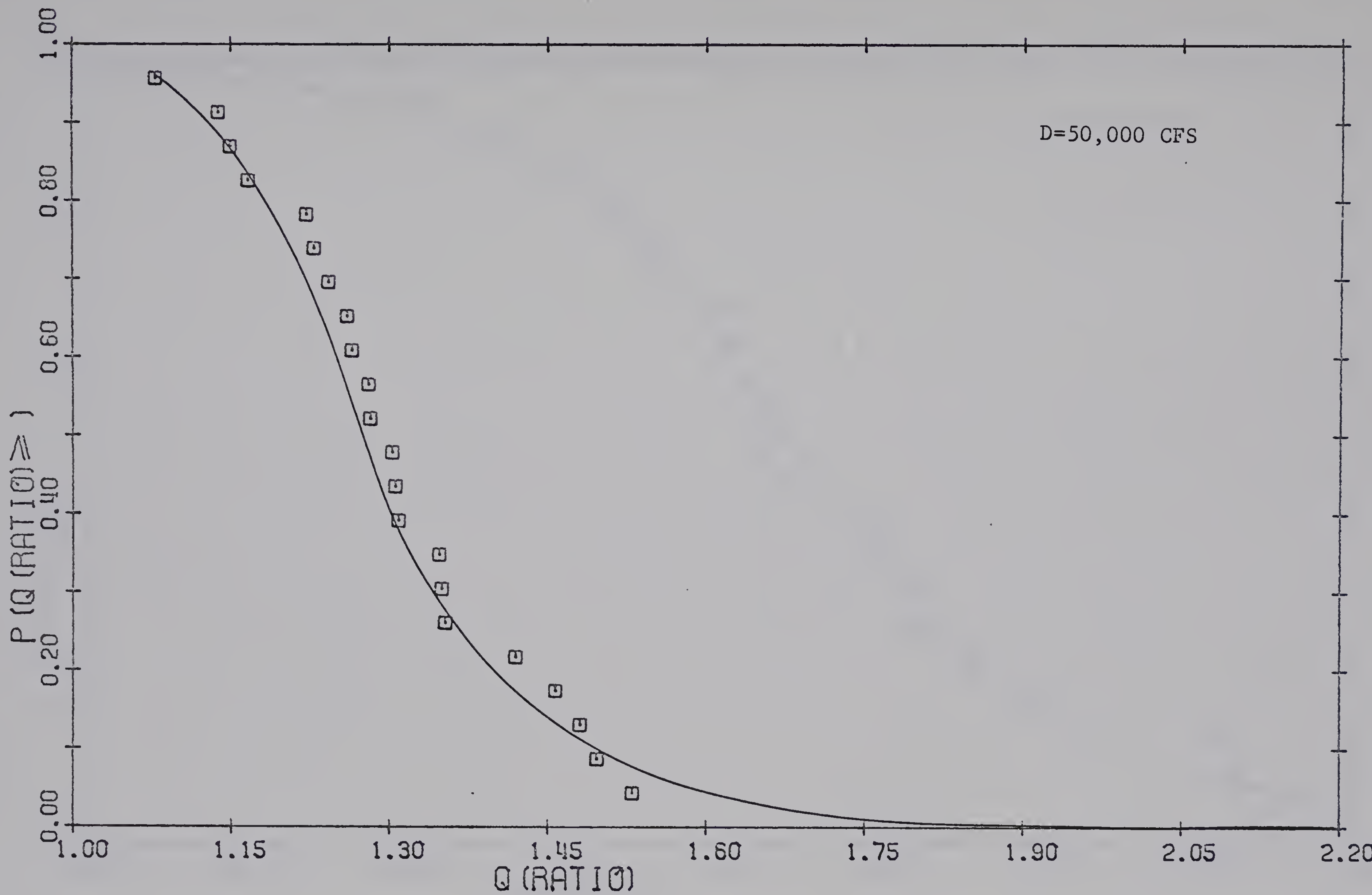


FIG. C-IIB PROBABILITY CURVE FOR Q (RATIO) : RECESSION CASES

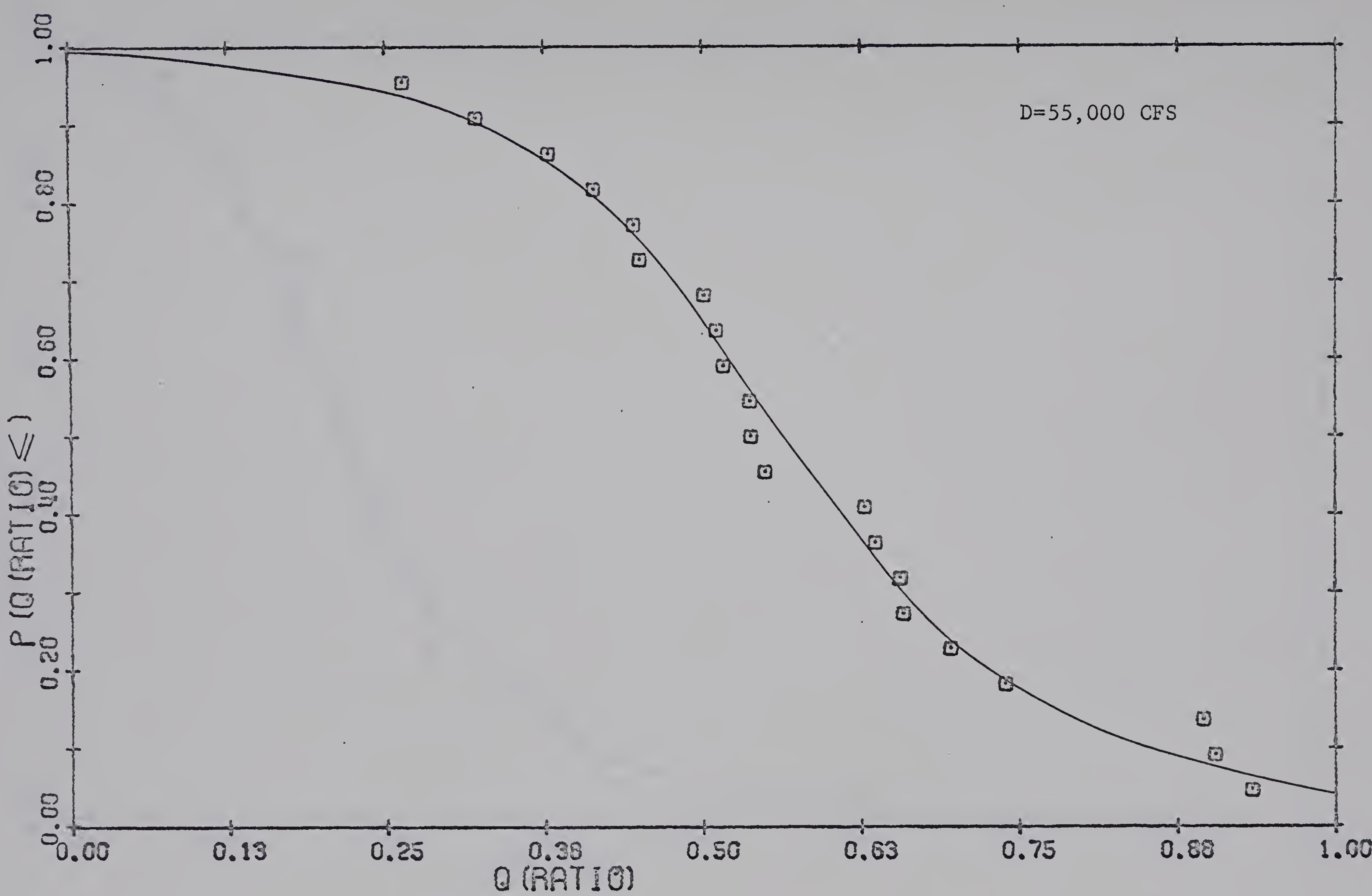


FIG. C-I2A PROBABILITY CURVE FOR Q (RATIO) : RISING CASES

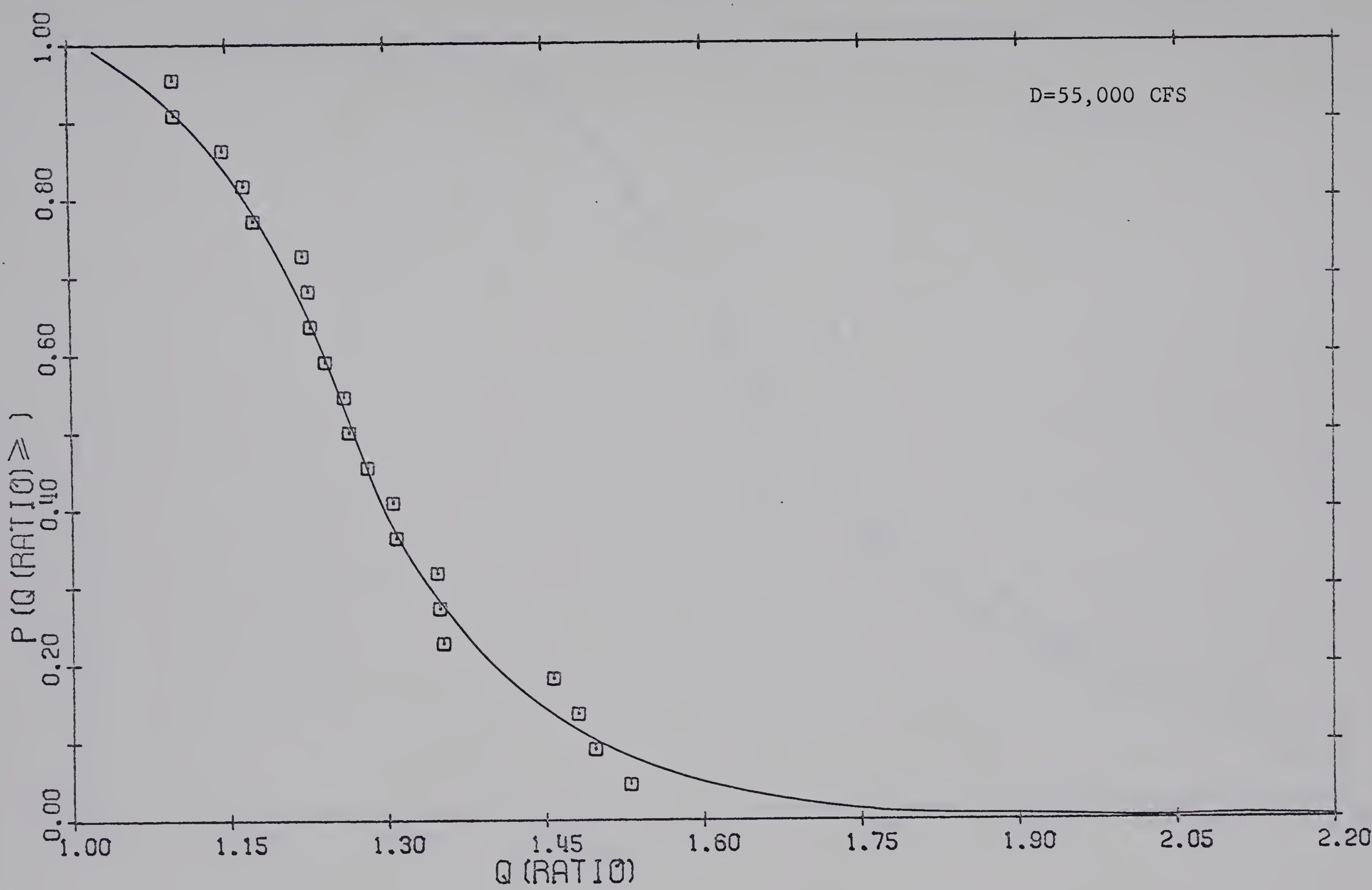


FIG. C-I2B PROBABILITY CURVE FOR Q (RATIO): RECESSION CASES

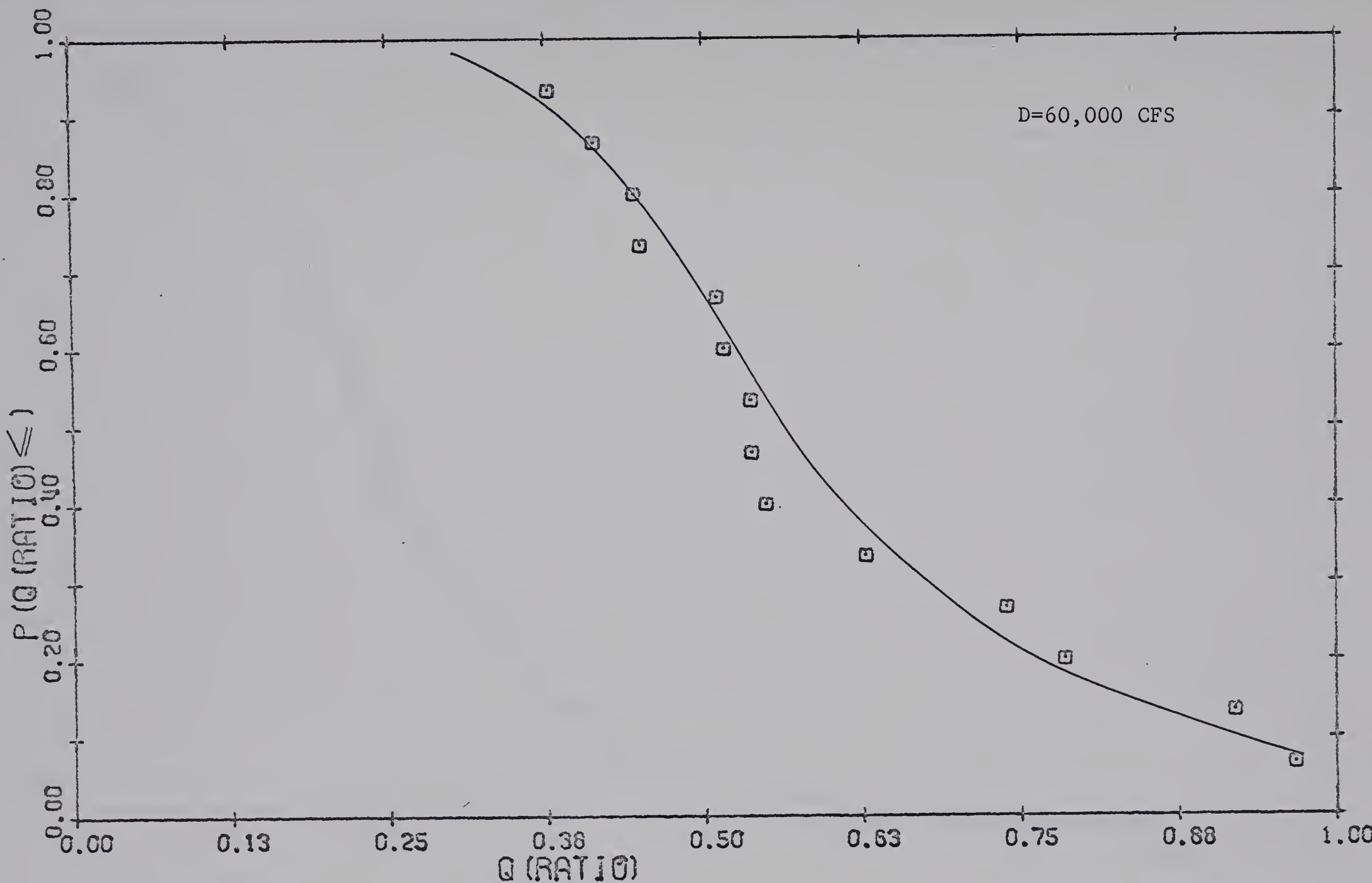


FIG. C-13A PROBABILITY CURVE FOR Q (RATIO): RISING CASES

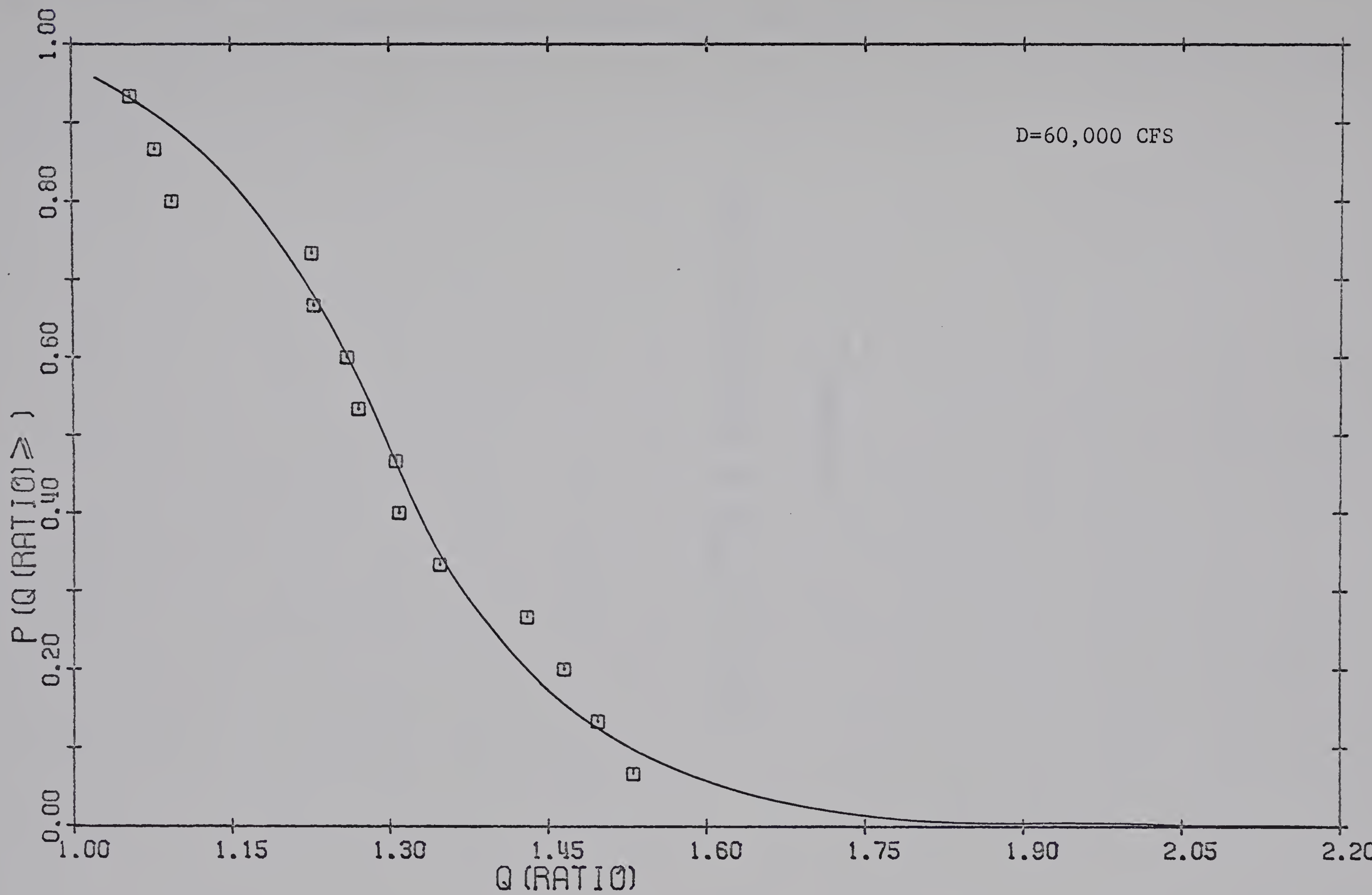


FIG. C-I3B PRØBABILITY CURVE FØR Q (RATIØ) : RECESSIØN CASES

APPENDIX D

TABLES FOR THE VALUES OF Q_{RATIO}, Q_1 and Q_2.

TABLE D.1A VALUES OF QRATIO : RISING CASES.

Probability	0.01	0.02	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	0.99
Discharge Level D CFS												
1000	0.40	0.53	0.72	0.82	0.86	0.89	0.92	0.93	0.95	0.97	0.98	0.99
5000	0.18	0.21	0.52	0.62	0.70	0.75	0.80	0.85	0.88	0.93	0.97	0.99
10000	0.20	0.25	0.55	0.65	0.73	0.78	0.83	0.85	0.89	0.92	0.95	0.95
15000	0.18	0.28	0.53	0.65	0.73	0.78	0.82	0.85	0.88	0.90	0.93	0.99
20000	0.27	0.29	0.46	0.59	0.68	0.75	0.79	0.85	0.88	0.91	0.93	0.98
25000	0.25	0.28	0.42	0.54	0.65	0.72	0.78	0.83	0.88	0.90	0.95	0.98
30000	0.25	0.26	0.40	0.49	0.56	0.64	0.70	0.76	0.80	0.85	0.94	0.98
35000	0.23	0.37	0.45	0.50	0.53	0.55	0.65	0.74	0.78	0.82	0.99	0.99
40000	0.20	0.26	0.43	0.47	0.53	0.55	0.64	0.70	0.74	0.78	0.85	0.99
45000	0.15	0.23	0.38	0.45	0.50	0.54	0.61	0.68	0.73	0.83	0.888	0.92
50000	0.15	0.22	0.33	0.43	0.45	0.50	0.54	0.60	0.64	0.69	0.75	0.90
55000	0.10	0.17	0.33	0.42	0.48	0.52	0.55	0.60	0.65	0.73	0.90	0.99
60000	0.10	0.29	0.40	0.45	0.48	0.52	0.54	0.59	0.68	0.79	0.93	1.00

TABLE D.1B. VALUES OF Q_{RATIO}: RECESSION CASES.

Probability	0.01	0.02	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	0.99
Discharge Level, D (CFS)												
1000	1.950	1.790	1.350	1.220	1.150	1.120	1.100	1.075	1.050	1.030	1.020	1.005
5000	1.650	1.590	1.300	1.190	1.125	1.100	1.085	1.070	1.055	1.040	1.025	1.010
10000	2.000	1.460	1.225	1.190	1.170	1.140	1.120	1.100	1.080	1.070	1.050	1.020
15000	1.900	1.600	1.270	1.225	1.175	1.160	1.140	1.120	1.100	1.0751	1.650	1.015
20000	1.900	1.500	1.280	1.230	1.175	1.160	1.140	1.125	1.100	1.075	1.050	1.020
25000	1.800	1.540	1.300	1.225	1.175	1.150	1.125	1.115	1.095	1.075	1.050	1.015
30000	1.800	1.640	1.320	1.230	1.205	1.185	1.170	1.140	1.120	1.095	1.060	1.020
35000	1.900	1.740	1.425	1.310	1.250	1.220	1.190	1.175	1.150	1.140	1.100	1.005
40000	2.000	1.780	1.450	1.350	1.300	1.250	1.210	1.175	1.140	1.110	1.085	1.050
45000	2.000	1.800	1.475	1.350	1.300	1.275	1.250	1.180	1.150	1.145	1.125	1.030
50000	2.100	1.900	1.500	1.425	1.350	1.300	1.280	1.260	1.240	1.175	1.140	1.050
55000	2.100	1.600	1.500	1.450	1.350	1.300	1.260	1.250	1.220	1.750	1.100	1.050
60000	2.500	1.700	1.520	1.450	1.375	1.325	1.275	1.250	1.225	1.175	1.100	1.050

TABLE D.2A

VALUES OF Q_1 AND Q_2: RISING CASES.

Probability / Discharge Level D CFS	0.01	0.02	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	0.99
1000	571	693	841	898	925	942	956	964	974	985	990	995
	1429	1307	1159	1102	1075	1058	1044	1036	1026	1016	1010	1005
5000	1525	1736	3421	3846	4118	4286	4444	4595	4667	4805	4911	4975
	8475	8264	6579	6154	5882	5714	5556	5405	5333	5195	5089	5025
10000	3333	4000	7097	7879	8439	8764	9041	9189	9418	9583	9744	9950
	16667	16000	12903	12121	11561	11236	10959	10811	10582	10416	10256	10050
15000	4468	6563	10328	11818	12558	13099	13471	14595	14000	14211	14456	14887
	25532	23437	19672	18182	17442	16901	16529	15405	16000	15789	15544	15113
20000	8504	8992	12603	14843	16120	17077	17654	18320	18610	19058	19275	19798
	31496	31008	27397	23157	23880	22923	22346	21680	21390	20942	20725	20208
25000	10000	10938	14789	18554	19697	20930	21831	22603	23334	23684	24293	24684
	40000	39063	35211	31446	30307	29070	28169	27397	26666	26316	25707	25316
30000	12000	12381	17143	19732	21539	23303	24706	25909	26667	27568	29072	29697
	48000	47619	42857	40268	38461	36697	35294	34091	33333	32432	30928	30303
35000	12857	14000	18905	21557	23333	24099	24839	27576	29770	30563	31539	34824
	57143	56000	51095	48443	46667	45901	45161	42424	40230	39437	38461	35176
40000	13334	16508	23662	25578	27541	28387	31220	32941	34023	35056	36756	39900
	66666	63492	56338	54422	52459	51613	48780	47059	45977	44944	43244	40100
45000	11739	16829	24546	27931	30000	31559	34100	36269	37826	40820	42128	43125
	78261	73171	65454	62069	60000	58441	55900	53731	52174	49180	47827	46875
50000	13040	18030	24810	29830	31040	33330	35070	37500	39030	40830	42860	47370
	86960	81970	75190	70175	68970	66660	64940	62500	60980	59170	57140	52630
55000	10000	15980	26980	32540	35420	37630	39030	41250	43330	46420	52110	54860
	100000	94020	83020	77460	74580	72370	70970	68750	66660	63580	57900	55140
60000	10910	26980	34290	37240	38640	41050	42080	44530	48360	52960	57660	59970
	109090	93020	85710	82760	81360	78950	77920	75470	71640	67040	62340	60030

TABLE D.2B

VALUES OF Q_1 AND Q_2: RECESSION CASES.

Discharge \ Probability	0.01	0.02	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	0.99
1000	1320	1280	1150	1090	1070	1050	1050	1030	1020	1010	1010	1000
	678	717	847	901	930	943	952	964	976	985	990	998
5000	6220	6130	5650	6430	5290	5230	5200	5160	5130	5090	5060	5020
	3770	3860	4340	4560	4700	4760	4790	4830	4860	4900	4930	4970
10000	13300	11800	11000	10800	10700	10600	10500	10400	10300	10300	10200	10090
	6660	8130	8980	9130	9210	9340	9430	9520	9620	9660	9760	9900
15000	19600	18400	16700	16500	16200	16100	15900	15800	15700	15500	15300	15100
	10300	11500	13200	13400	13700	13800	14000	14100	14200	14400	14600	14800
20000	26200	24000	22400	22000	21600	21400	21300	21100	20900	20700	20400	20100
	13700	16000	17500	17900	18300	18500	18600	18800	19000	19200	19500	19800
25000	32100	30300	28200	27500	27000	26700	26400	26300	26100	25900	25600	25100
	17800	19600	21700	22400	22900	23200	23500	23600	23800	24000	24300	24800
30000	38500	37200	34100	33000	32700	32500	32300	31900	31600	31300	30800	30200
	21400	22700	25800	26900	27200	27400	27600	28000	28300	28600	29100	29700
35000	45800	44400	41100	39600	38800	38400	38000	37800	37400	37200	26600	35000
	24100	25500	28800	30300	31100	31500	31900	32800	32500	32700	33300	34900
40000	53300	51200	47300	45900	45200	44400	43800	43200	42600	42000	41600	40900
	26600	28700	32600	34000	34700	35500	36200	36700	37300	37900	38300	39000
45000	59900	57800	53600	51700	50800	50400	49900	48700	48100	48000	47600	45600
	30000	32100	36300	38300	39100	39500	40000	41200	41800	41900	42300	44300
50000	67700	65500	60000	58700	57400	56500	56100	55700	55300	54000	53200	51200
	32200	34400	40000	41200	42500	43400	43800	44200	44600	45900	46700	48700
55000	74500	67600	66000	65100	63100	62100	61300	61100	60400	59400	58200	56300
	35400	42300	44000	44800	46800	47800	48600	48800	49500	50500	51700	53600
60000	85700	75500	72300	71000	69400	68300	57200	66600	66000	64800	62800	60800
	34200	44400	47600	48900	50500	51600	2700	53300	53900	55100	57100	59100

APPENDIX E

TABLES FOR THE STUDY OF THE EFFECT OF RECORD LENGTH.

TABLE E.1A

EXTREME VALUES FOR MEDIANS OF Q_{ratio}: RISING CASES

Discharge Level CFS / Record Length Years	1000	10000	20000	30000	40000	50000	60000
10	0.962	0.853	0.846	0.777	0.766	0.636	0.728
	0.878	0.784	0.694	0.572	0.512	0.235	0.235
15	0.941	0.845	0.843	0.749	0.711	0.636	0.549
	0.884	0.801	0.699	0.636	0.505	0.429	0.429
20	0.933	0.834	0.837	0.749	0.658	0.592	0.549
	0.899	0.813	0.733	0.639	0.555	0.429	0.445
25	0.934	0.831	0.826	0.732	0.658	0.636	0.549
	0.904	0.804	0.750	0.666	0.557	0.445	0.447
30	0.916	0.824	0.797	0.729	0.658	0.547	0.543
	0.905	0.804	0.751	0.673	0.555	0.450	0.480
35	0.920	0.831	0.796	0.718	0.656	0.547	0.543
	0.905	0.805	0.776	0.666	0.557	0.450	0.480
40	0.920	0.826	0.794	0.696	0.638	0.542	0.526
	0.907	0.821	0.772	0.677	0.636	0.526	0.513
45	0.920	0.825	0.793	0.696	0.638	0.537	0.537
	0.911	0.817	0.790	0.686	0.636	0.537	0.526
48	0.911	0.821	0.793	0.700	0.638	0.537	0.537

TABLE E.1B

EXTREME VALUES FOR MEDIANS OF Q_{ratio}: RECESSION CASES

Discharge Level CFS / Record Length Years	1000	10000	20000	30000	40000	50000	60000
10	1.159	1.163	1.158	1.209	1.283	1.378	1.429
	1.049	1.093	1.120	1.111	1.180	1.156	1.181
15	1.140	1.139	1.152	1.209	1.269	1.378	1.429
	1.056	1.095	1.132	1.140	1.180	1.156	1.181
20	1.123	1.138	1.151	1.201	1.267	1.308	1.429
	1.081	1.105	1.132	1.152	1.201	1.212	1.270
25	1.119	1.134	1.149	1.197	1.267	1.308	1.369
	1.081	1.110	1.136	1.161	1.201	1.250	1.259
30	1.119	1.134	1.149	1.187	1.255	1.308	1.327
	1.086	1.113	1.142	1.163	1.201	1.269	1.270
35	1.111	1.120	1.149	1.177	1.255	1.292	1.327
	1.086	1.115	1.142	1.164	1.208	1.269	1.288
40	1.105	1.119	1.145	1.166	1.240	1.281	1.307
	1.089	1.116	1.143	1.163	1.209	1.280	1.270
45	1.093	1.118	1.145	1.164	1.240	1.281	1.307
	1.090	1.117	1.143	1.162	1.225	1.281	1.288
48	1.093	1.118	1.144	1.162	1.240	1.292	1.288

TABLE E.2A

PERCENT DEVIATIONS FOR EXTREMES OF Q_{RATIO}-MEDIANS: RISING CASES

Discharge Level (CFS) / Record length years	1000	10000	20000	30000	40000	50000	60000
10	5.55	3.84	6.59	11.13	20.15	18.35	35.58
	-3.65	-4.48	-12.49	-18.19	-19.68	-56.19	-56.19
15	3.25	2.86	6.29	7.04	11.47	18.35	2.13
	-2.95	-2.46	-11.83	-9.09	-20.77	-20.05	-20.05
20	2.38	1.55	5.56	7.04	3.15	10.24	2.13
	-1.34	-1.02	-7.57	-8.60	-12.93	-20.05	-17.12
25	2.48	1.20	4.11	4.68	3.15	18.35	2.13
	-0.77	-2.05	-5.51	-4.78	-12.62	-17.12	-16.68
30	0.47	0.38	0.41	4.14	3.15	1.76	1.46
	-0.73	-2.05	-5.37	-3.80	-12.93	-16.25	-10.64
35	0.97	1.20	0.38	2.58	2.86	1.76	1.46
	-0.68	-1.94	-2.90	-4.78	-12.62	-16.25	-10.64
40	0.95	0.60	0.06	0.56	0	0.91	2.00
	-0.53	-0.06	-2.72	-3.23	-0.32	-2.00	-4.48
45	0.94	0.45	0	0.56	6	0.06	0
	0	-0.44	-0.38	-1.90	-0.32	-0.06	-2.00
48	0	0	0	0	0	0	0

TABLE E.2B

PERCENT DEVIATIONS FOR EXTREMES OF Q_{RATIO} - MEDIANS: RECESSION CASES

Discharge Level CFS / Record Length Years	1000	10000	20000	30000	40000	50000	60000
10	6.02	3.99	1.25	10.40	3.48	6.68	11.00
	-4.02	-2.22	-2.02	-4.34	-4.76	-10.52	-8.24
15	4.30	1.84	0.76	4.06	2.35	6.68	11.00
	-3.33	-2.09	-1.03	-1.86	-4.76	-10.52	-8.24
20	2.71	1.74	0.69	3.33	2.18	1.27	11.00
	-1.11	-1.24	-1.03	-0.86	-3.09	-6.12	-1.37
25	2.34	1.37	0.46	3.03	2.18	1.27	6.29
	-1.07	-0.79	-0.65	-0.03	-3.09	-3.17	-2.19
30	2.34	1.37	0.46	2.16	2.22	1.27	3.08
	-0.67	-0.53	-0.13	-0.07	-3.09	-1.71	-1.37
35	1.61	0.17	0.45	1.27	1.23	0	3.08
	-0.59	-0.32	-0.09	-0.18	-2.50	-1.71	0
40	1.10	0.03	0.16	0.36	0	0.81	1.48
	-0.46	-0.21	-0.03	-0.07	-2.43	-0.87	-1.37
45	0	0.03	0.16	0.18	0	0.81	1.48
	-0.22	-0.10	-0.03	0	-1.21	-0.81	0
48	0	0	0	0	0	0	0